CHAPTER 5 SUPPORTING LIFELINES

Lifelines are key facilities and utility systems which are vital to the operation of a terminal. They include fire detection and suppression, electric power, gas and liquid fuels, telecommunications, transportation and water supply and sewers. The following explains the development of the proposed design criteria and gives current procedures and all relevant codes. Observed damage from previous earthquakes was analyzed to develop failure modes, from which a design criteria was produced.

Potential problems facing a terminal after an earthquake are building structural failures, damage to waterfront retaining structures, tank failures, crane failures, utilities disruption and hazardous materials spills. Typical lifeline problems involve above ground and underground pipeline breaks from soil movement, collapse of pipelines caused by failed supports, shifting of tanks on their foundations, and buckling of tanks. Related factors which add to the complexity of recovery are the dislodging of asbestos or encapsulated asbestos insulation; industrial equipment damage caused by sliding or overturning, or internal failures; falling containers of hazardous materials which may rupture and impede recovery. Other factors can complicate the ability to respond to these releases, including: lack of water for washing down spills, disruption in communication, closure of roads, and lack of transportation access routes.

While guidance can only be given in a general form since specific circumstances control each case, the Resource Conservation and Recovery Act as a public law establishes mandates concerning the pollution of the environment and as such has direct relevance to this criteria. It sets a high seismic design level at which municipal waste facilities are to function to preclude contamination of the environment. This law requires that we place a high value on ground water and preclude contamination. As such this is probably the controlling relevant guidance for non-nuclear polluting or hazardous materials.

Essential Vs. Ordinary Construction

When considering a facility supporting an essential function, it is critical that the facility be considered as a system. It is not sufficient to consider a facility simply as a building structure, but rather it is required to consider all the elements required to accomplish the mission to be accomplished in that structure. This usually includes requirements for fire detection and suppression, electrical power, mechanical systems, water and sewer, communications, road access etc.

Seismic Codes Related To Lifelines

Current seismic codes when viewed as an ensemble, form a basis for understanding the state-of-the-art of risk quantification and the engineering profession's determination of prudent action. This section summarizes a number of seismic standards. DOE procedures and The Resource Conservation and Recovery Act (40 CFR 248 -USEPA 1991) both directly consider hazardous materials.[1]

1994 Uniform Building Code- The building code has been one of the origins for lifeline design under the category of non-building structures. The pseudostatic approach calculates an equivalent lateral force, V as;

$$V = [\ (ZIC) / R_w\ ] * W$$

$$C = 1.25\ S / T^{2/3}$$

where

T	Structural period < 2.75 seconds
Z	Zone factor
I	Importance factor = 1.0 to 1.25
S	Soil factor
R_w	Response modification factor or ductility factor

The Z factor represents the design earthquake ground acceleration according to the zone in which the structure is located and has a 10 percent probability of exceedance in 50 years. This is nominally a 500 year event or an event with an annual probability of exceedance of 2×10^{-3}. This design load is modified by the other factors of the equation; performance drift limits are used. The importance factor is used to increase the design load for important structures; however, the 1.25 is not large enough to produce elastic response under a severe earthquake. Wen et al (1994) notes that "this small range (in I) is hard to justify since the uncertainty in the seismic excitation is generally so large that the different reliability levels required of the structure would lead to a much larger range of the structural resistance. To determine the importance factor rationally and qualitatively, a calibration of this value needs to be performed according to the performance goal required of the structure in terms of acceptable risks of limit states." The C factor is a function of the site soil conditions and the fundamental period of the structure.

The R_w factor allows for ductility in typical building structures and is also used for non-building elements. For non-buildings UBC Table 16-Q specifies R_w such as 3.0 for tanks. The ratio of C/ R_w shall not be less than 0.5 The provisions call for computation of the lateral force of the tank using the entire weight of the tank and its contents. A response spectra approach allowing for inertial effects of the contents is permitted.

Lateral force on elements and components shall be designed for:

[1] The following sections contain various code provisions. The nomenclature and notation of the original reference was kept the same and is not necessarily consistent among references.

$$F_p = Z\ I\ C_p\ W_p$$

where C_p is specified in Table 16-O for elements such as tanks, racks, anchorage, plumbing etc. and W_p is the weight of the element. Rigid elements are designed for 0.5 of their weight times the Z and I factors.

For equipment in facilities drift must be checked. Drift limits are specified in terms of the interstory displacement divided by story height, d, as:

$$d = 0.03/R_w \text{ and } < 0.004$$

Wen et al (1994) notes the drift is about 1.5 percent independent of the response modification factor. This is not consistent with a reliability based approach.

1997 Uniform Building Code – The 1997 Uniform Building Code is a transitional code going from the 1994 UBC to a national building code based on the NEHRP guidelines expected in the year 2000. Section 1632 present equations for calculating horizontal forces on nonstructural components and equipment.

1992 - 1995 NEHRP Provisions- The National Earthquake Hazard Reduction Program (NEHRP) has been used in waterfront design. The design earthquake is established as 10 percent chance of exceedance in 50 years which may result in both structural and non-structural damage which is expected to be repairable. For larger motions the intent is to preclude collapse. Peak ground acceleration maps are provided. The 1992 provisions computed seismic shear as:

$$V = C_s\ W$$

$$C_s = (1.2\ A_v\ S) / (R\ T^{2/3})$$

$$\text{but} \qquad C_s = < (2.5\ A_a) / R$$

where A_a and A_v are defined as effective peak acceleration and effective peak velocity-related acceleration. R is the response modification factor similar to the UBC but with different values. The 1994 provisions modified the equation by introducing amplification factors, F_a and F_v, and redefined the soil types into six groups:

$$C_s = (1.2\ A_v\ F_v) / (R\ T^{2/3})$$

$$\text{but} \qquad C_s = < (2.5\ A_a\ F_a) / R$$

The 1995 NEHRP soil site classes which establish values for F_a and F_v are defined as:

A) Hard rock with measured shear wave velocity, $v_s >$ 5,000 ft/sec (1,500 m/s)

B) Rock with 2,500 ft/sec < v_s < 5,000 ft/sec (760 m/s < v_s < 1500 rn/s)

C) Very dense soil and soft rock with 1,200 ft/sec < v_s < 2,500 ft/sec
360 m/s < v_s 760 rn/s) or with either N > 50 or s_u 2,000 psf (100 kPa)
where N is average blow count SPT and s_u is average undrained shear strength

D)Stiff soil with 600 ft/sec < v_s < 1,200 ft/sec (180 rn/S < v_s < 360 rn/s)
or with either 15< N < 50 or 1,000 psf < s_u < 2,000 psf (50 kPa < s_u < 100 kPa)

E) A soil profile with v_s < 600 ft/sec (180 m/s) or any profile with more than 10 ft
(3 m) of soft clay defined as soil with PI > 20,*w* > 40 percent, and S_u < 500 psf (*25* kPa)

F) Soils requiring site-specific evaluations:

1.Soils vulnerable to potential failure or collapse under seismic loading such as
liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils.

2. Peats and/or highly organic clays (H > 10 ft (3 rn) of peat and/or highly organic
clay where H = thickness of soil)

3.Very high plasticity clays (H > 25 ft (8 m) with PI > 75)

4.Very thick soft/medium stiff clays, H> 120 ft (36 m)

EXCEPTION: When the soil properties are not known in sufficient detail to determine the Soil Profile Type, Type D shall be used. Soil Profile Types E or F need not be assumed unless the regulatory agency determines that Types E or F may be present at the site or in the event that Types E or F are established by geotechnical data.

The 1995 NEHRP provides the following steps for classifying a site.

Step 1: Check for the four categories of Soil Profile Type F requiring site-specific evaluation. If the site corresponds to any of these categories, classify the site as Soil Profile Type F and conduct a site-specific evaluation.

Step 2: Check for the existence of a total thickness of soft clay > 10 ft (3 m) where a soft clay layer is defined by: Su < 500 psf (25 kPa), w > 40 percent, and PI > 20. If these criteria are satisfied, classify the site as Soil Profile Type E.

Step 3: Categorize the site using one of the following three methods with v_s, N, and s_u

a. v_s for the top 100 ft (30 m)

b. N for the top 100 ft (30 m)

c. N_{ch} for cohesionless soil layers (PI < 20) in the top 100 ft (30 m) and average s_u for cohesive soil layers (PI > 20) in the top 100 ft (30 m) where N_{ch} is average blowcount for cohesionless layers from SPT

The NEHRP provisions found in FEMA 223A Section 3.3.9 discusses storage tanks and allows either the AWWA or the API procedures. It specifies that pipe connections to steel storage tanks provide for 2 inches of vertical displacement for anchored tanks and 12 inches for unanchored tanks. It further specifies piping systems to be made of ductile materials; design strengths for service load combinations may be 90 percent of yield strength for ductile steel, aluminum, or copper, 70 percent of yield strength for threaded pipe made from ductile material and 25 percent of minimum tensile strength for plastic pipe. Threaded connections in piping constructed of nonductile materials shall not more than 20 of minimum specified tensile strength. Section 3.1.3 defines seismic force levels for tanks and piping.

1997 NEHRP/FEMA 273 - FEMA 273, ‘ NEHRP Guidelines for the Seismic Rehabilitation of Buildings” defines basic safety objectives by two earthquake levels, 10 percent probability of exceedance in 50 years and 2 percent probability of exceedance in 50 years. The first level treats life safety while the second addresses collapse prevention. Chapter 11 discusses nonstructural elements. The chapter presents two approaches for nonstructural rehabilitation. The first is a prescriptive procedure where published standards are used. The second procedure is an analytical procedure horizontal component forces are computed based on the spectra level, performance objective and component weight.

1997 NEHRP/FEMA 302 – FEMA 302 addresses design of new buildings and further develops the previous work. A set of maps are used to determine ground motion parameters, which are adjusted based on soil classes discussed above. The ground motion is taken as two-thirds of the 2500 year maximum considered earthquake. Response modification factors, R, system overstrength factor, Ω_o, and deflection amplification factor C_d are used. The basic form of the base shear equation is:

$$V = C_s W$$

where

$$Cs = S_{DS} / (R/I)$$

where

I The occupancy importance factor
R The response modification factor

The determination of horizontal forces on nonstructural elements and equipment is essentially the same as in FEMA 273.

DOE Criteria -The Department of Energy (DOE) guidelines define four classes of structure, General Use, Important / Low Hazard, Moderate Hazard, and High Hazard. The later two classes refer to nuclear facilities. This work establishes a risk acceptability criteria which has direct correlation to the containment of hazardous materials and lifelines. General Use facilities are typical ordinary structures to be designed by current code provisions. Important / Low Hazard facilities would include laboratories, computer centers, hazard recovery facilities and other facilities with a building code importance factor of 1.25. Moderate Hazard facilities include facilities where confinement of contents is necessary to protect personnel including the handling of radioactive and toxic materials. High Hazard facilities include facilities where confinement of contents is necessary for public and environmental protection such as nuclear facilities; these facilities represent hazards with potential long term and widespread effects. Specification of the design earthquake is established in terms of the annual probability of exceedance starting at a value similar to the UBC value and decreasing with class of structure. The annual probabilities of exceedance values expressed as earthquake nominal return times used for the four classes of structure are:

Structure Class	Earthquake Return Time (years)	Annual Exceedance Probability
General Use	500	2×10^{-3}
Low Hazard	1000	1×10^{-3}
Moderate Hazard	1000	1×10^{-3}
High Hazard	5000	2×10^{-4}

The DOE guidelines define the performance goals of each class of structure. Associated with each performance goal is a probability of the structural system meeting that goal. The following table shows that relationship.

Structure Class	Performance Goal	Probability of failure to meet goal
General Use	Occupant Safety Prevent major structural damage/collapse Code provisions	0.5
Low Hazard	Continued Operation Capacity to function, occupant safety, relatively minor structural damage	0.5
Moderate Hazard	Continued Functionality Hazard Confinement Limited damage to insure containment of hazardous materials	0.1
High Hazard	Continued Functionality Hazard Confinement Limited damage to insure containment of hazardous materials	0.05

By combining the earthquake probability of occurrence and the probability of exceedance of failure shown above the annual probability of failure can be calculated and is shown below. These values range from 0.001 for general use structures in which the measure of performance is probability of collapse to 0.00001 for high hazard facilities in which the measure of performance is failure of containment of the high hazard. The goals and probabilities are:

Structure Class	Performance Goal	Annual Exceedance Probability of Failure
General Use	Occupant Safety	1×10^{-3}
Low Hazard	Continued Operation	5×10^{-4}
Moderate Hazard	Continued Functionality Hazard Confinement	1×10^{-4}
High Hazard	Continued Functionality Hazard Confinement	1×10^{-5}

This data is thought to be a significant statement on the general acceptability of seismic risk and as such has direct bearing on establishing guidance for comparable operations associated with oil terminal facilities.

Navy Criteria- The Navy uses NAVFAC P355 for the design of buildings and associated details. Provisions are included in this reference for design of lifeline supports and equipment using 1990 SEAOC lateral force criteria. Chapter 11 presents a procedure for designing architectural elements. Chapter 12 addresses mechanical and electrical component anchorage while Chapter 13 deals with non-buildings and addresses tank design criteria. Chapter 14 gives an overview of utility systems and pipe details. The NAVFAC P355 (1992) does not reflect the most recent UBC and NEHRP provisions and is undergoing revision to conform to NEHRP.

The general lateral base shear applied to a structure is the product of the structure weight , a zone coefficient, an importance coefficient, and a site factor composed of soil type and structure period, all divided by a ductility factor based on the type of lateral force resisting system. The pseudostatic load is distributed along the height of the structure and resulting stresses and overturning moments determined. Combinations of dead load, live load and other loads are used and orthogonal horizontal loads are combined to produce a total. Drifts are checked. Allowable stress design is used with adjustment factors.

The general, code anchorage force to be applied to a structure for relatively small elements of equipment within a building is specified as the product of the equipment weight, a zone factor, an importance factor, and a factor describing the type of element. The element is limited to less than 10 percent of the total weight of the structure or to 20 percent of the floor weight at the element level. Drift limitations can apply. For self-supported equipment on the ground, the value of F_P may be reduced by 2/3. Large elements are designed as non-buildings using the provisions of Chapter 13. Pipes containing hazardous materials within a building require special provisions for flexibility such as, flexible couplings, expansion joints, and spreaders.

40 CFR 248 -USEPA 1991 The Resource Conservation and Recovery Act - This public law specifies the design requirements for municipal solid waste landfills (MSWLF). There is major concern that these dumps can pollute the ground water. The law states “new MSWLF units and lateral expansions shall not be located in seismic impact zones unless... all containment structures, including liners, lechate collection systems and surface control systems are designed to resist the maximum horizontal acceleration in lithified earth material for the site” The law mandates that a composite liner composed of a geomembrane and 2 feet of low permeability soil be used. The maximum acceleration is defined as emanating from a seismic event with a 90 percent chance of not being exceeded in 250 years; this is nominally a 2500 year return time event. Design criteria is given for allowable concentrations of toxic chemicals and acceptable values of hydraulic conductivity.

This legislation is a significant statement which establishes defined risk limits for seismic pollution of the environment and as such is applicable to comparable oil terminal facilities.

American Water Works Association D100, D103, D110 Standards. These standards describe the design of bolted and welded steel tanks and prestressed concrete tanks. Structures to be designed for Seismic Zones 1, 2 or 3 may be designed for a fixed percentage weight of 2.5 percent, 5 percent and 10 percent respectively. Elevated tanks are design using:

$$V = Z\ KC\ SW$$

where

$$C = 1/(15\sqrt{T})$$

Specific values for K are given.

Tanks on the ground in Seismic Zone 4 require a pseudostatic design but allow for a response spectra. The horizontal base shear is given by:

$$V = Z\ I\ K\ S\{\ C_1\ (W_s + W_r + W_1) + C_2\ W_2\ \}$$

and the overturning moment is given by:

$$M = Z\ I\ K\ S\{\ C_1\ (W_s X_s + W_r H_t + W_1 X_1) + C_2\ W_2 X_2\ \}$$

Where

C_1	Factor based on natural period
C_2	Factor based on natural period
H_t	Total height of tank shell
I	Importance factor
K	Structure coefficient depends on type and anchorage
S	Soil factor
W_r	Weight of effective mass of tank roof
W_s	Weight of effective mass of tank wall
W_1	Weight of effective mass of tank contents moving with tank shell
W_2	Weight of effective mass of first mode tank sloshing
X_s	Height from bottom of tank shell to cg of shell
X_1	Height from bottom of tank shell to centroid of lateral force applied to W_1
X_2	Height from bottom of tank shell to centroid of lateral force applied to W_2
Z	Zone factor

The bolted steel tank standard uses an SC_1 value of 0.14. The fundamental period of the tank is prescribed by equation and varies depending on the particular standard and tank type. A fundamental period for the sloshing mode is also computed. The K value varies with the type of tank and whether it is anchored or not. Unanchored tanks have higher K values.

Response spectra values can be substituted for equation values. The approach considers that the loading consists of components at the tank fundamental frequency and also components at the sloshing frequency. Response spectra values based on a tank period can be substituted for $ZIKSC_1$. Additionally, sloshing period values can be substituted for $ZIKSC_2$. Vertical force components can be included in the computation. The designer has the option to compute the resultant separately or in conjunction with horizontal forces. Tank wall stresses are computed from overturning moments and compared with allowable values. Formulas are given for computation of vertical compressive and tensile forces at the tank base. Flat-bottom tanks may be anchored or unanchored. Where tanks are unanchored the maximum thickened annular ring width at the base used to limit overturning is limited to 7 percent of the tank radius and the thickness shall not exceed the thickness of the shell thickness at the bottom. Anchored tanks could be susceptible to tearing if not properly designed. Hydrodynamic seismic tensile membrane forces are computed. Allowable stresses are increased by one-third for seismic forces. Guidelines are given for important foundation considerations including allowable bearing and the need for soil homogeneity across the foundation. Various types of tank foundations are discussed. The user shall specify the amount of tank freeboard for sloshing. Failure to provide for sloshing will damage the roof if the tank is completely full. Provisions are included to allow for local site conditions. A 2 percent damped curve is recommended for design of the structure and a 0.5 percent damped curve is recommended for sloshing of the liquid. The amplified acceleration shall be determined for the cantilever beam period of the shell and effective portion of the contained fluid. When site response return times are not given a maximum credible event or 10,000 year return time event can be used with a response reduction factor not to exceed 2.6.

The AWWA has standards for ductile iron, steel, concrete, and asbestos pipe; however they do not address seismic design directly.

American Petroleum Institute Standard 650- The American Petroleum Institute provisions follow 1980's code design and was revised and updated as recently as 1996. The tank overturning moment is:

$$M = Z\,I\,\{\,C_1\,(W_s X_s + W_r H_t + W_1 X_1) + C_2 W_2 X_2\,\}$$

where the terms are the same as defined for the AWWA equation above. The term C_1 is set at 0.60 unless the product of $Z\,I\,C_1$ and $Z\,I\,C_2$ are determined from response spectra. The term C_2 is defined by:

$$C_2 = 0.75\ S / T \qquad \text{for } T <= 4.5$$

$$C_2 = 3.375\ S / T^2 \quad \text{for } T > 4.5$$

If a spectrum is used for the factor Z I C_1, it should be developed for a damping coefficient of 2 percent of critical. The spectrum for the factor Z I C_2 should be based on the spectrum for Z I C_1 but with a damping coefficient of 0.5 percent of critical.

> Summers (1997) reports that extensive experimental studies and observations during past earthquakes have demonstrated that the radial length of uplifted bottom plate, and hence, the actual liquid weight resistance which is mobilized during an earthquake is underestimated by the API uplift model. He explains the reasons for this are that the API model does not account either for the in-plane stress in the bottom plate, or for the dynamic nature of the tank response. The API model also calculates a somewhat narrow compression zone at the toe of the tank, thus leading to large compressive stresses in the tank shell for relatively low overturning moments. Finally, the API approach does not account for the effect of foundation flexibility on the tank wall axial membrane stress distribution. These factors err on the conservative side and result in overdesign. The API procedure is recognized as a conservative approach and is acceptable for new tank design.

40 CFR 112: 38FR 34164 Environmental Protection Agency Regulations On Oil Pollution Prevention This public law applies to oil storage or processing facilities which are potential pollution sources. It does not apply to facilities where the storage capacity is 1,320 gallons or less and no single tank has a capacity in excess of 660 gallons. For facilities falling under provisions of this law, appropriate secondary containment is mandated such as dikes, curbs, sumps or ponds.

State of California Above Ground Storage Act of 1991 This law applies to sites containing petroleum/hazardous material storage tanks where the above ground storage capacity is over 1,320 gallons or where a single tank exceeds a capacity of 660 gallons. The law requires inspections, licensing and monitoring. The foundation system must be designed to allow for early detection of releases of materials before reaching the ground water.

American Railway Engineering Association, Chapter 9 Seismic Design For Railway Structures The procedure specifies three levels of ground motion: A Level I ground motion has a reasonable probability of being exceeded during the life of the structure and the structure is at a serviceable limit state which requires it to remain elastic. Only moderate damage which does not affect trains at restricted speeds is allowed. Allowable stresses are increased 150 percent in steel and 133 percent in concrete elements. The return period for a Level I earthquake is between 50 and 100 years. The determination of a specific ground motion level is left to the designer based on the type and volume of traffic expected. A Level II ground motion has a low probability of being exceeded during the life of the structure and represents a limit state to ensure overall structural integrity. The structure may respond in the inelastic range but ductilities are limited. The return period for a Level II earthquake is between 200 and 500 years. The selection of the specific level is left to the designer based on overall economics considering structure cost and train schedules. A Level III ground motion is established for a rare intense earthquake which establishes a survivability limit state which allows extensive damage but precludes collapse. Foundation failures are limited so as not to cause major changes in the structure geometry. The return period for a Level III earthquake is between 1000 and 2500 years. The selection of a specific level is left to the designer based the consequences of loss of the structure and include costs of construction, loss of use, existence of alternate routes and location of the bridge. Pseudostatic, spectral and dynamic procedures are used depending on the type and irregularity of the structure. The nominal 100 year, 500 year and 2500 year return time peak horizontal rock accelerations are specified on a national map

Standard Specification For Highway Bridges, AASHTO This is a national code and as such divides the US into regions based on levels of expected ground motion. A map is provided which shows peak horizontal rock accelerations with a 90 percent probability of not being exceeded in 50 years which is a nominal 500 year return time event. Two categories of bridge structure are defined, essential bridges which are expected to function after a design earthquake and other bridges which are designed for near elastic response at moderate events and for limited damage at the maximum credible event. Four categories A through D are defined to treat importance and variation in seismic acceleration potential. A and B are low treat level requirements while D is highest representing an essential structure in the highest exposure zone. Three site profiles are defined and serve to define site amplification. Elastic earthquake lateral forces are determined based on the map accelerations and site soil factor. Component response modification factors are used to reduce the elastic forces for substructure elements while connections of superstructure to abutment and expansion joints are increased. The modification factors are analogous to ductility factors. It is assumed that columns will yield when subjected to forces from the design ground motion but that the connection will be able to resist the deformations with little damage. Wall piers have minimal ductility and an R value of 2 was assigned. Well designed columns in a multi-column bent have good ductility and a value of 5 was assigned to them. Single columns lack redundancy thus a value of 3 was assigned. For C and D bridges the connections are designed for the maximum forces that can be developed by plastic hinging in the columns. The probability of elastic force levels not being exceeded in 50 years is in the range of 80 to 90 percent. Procedures are given to calculate displacements. Modal response techniques are used in the analysis of response. It is suggested that a factor of safety against liquefaction be 1.5 for important bridges. Guidance is given for pile design

1990 CALTRANS CALTRANS criteria was developed for non-buildings and is of general interest. It is summarized as:

$$V = ARS \; W / Z$$

where W is the total weight and Z is an adjustment factor for ductility and risk and based on the period and type of structural element. ARS is the 5 percent elastic response spectrum at the site in g's based on the maximum expected acceleration at bedrock or rocklike material. The seismic force in two directions is required and to be evaluated by adding 30 percent of the force to the component in the perpendicular direction. For conservative design, the vector sum can be used. A load factor of 1.0 is used and live load is not included. The strength reduction factor, ϕ, for concrete columns can be increase from 0.9 to 1.2 to recognize an increase in strength from well confined concrete.

Japan Gas Association Recommended Practice For Pipelines- The 1978 Miyagiken-oki earthquake caused heavy damage to the gas distribution system in Sendai City. Damage was concentrated in threaded steel pipelines of about 2-inch diameter. As a result of this guidelines were developed for Japan. Japan is divided into four seismic zones and three soil classifications are used. The seismically induced horizontal ground deformation is estimated by:

$$U = \alpha_1 \alpha_2 \; U_0$$

where

α_1 Constant based on site location in the range of 0.4 to 1.0
α_2 Constant based on soil condition and importance in the range of 0.5 to 1.8
U_0 Constant which is set at 5 centimeters

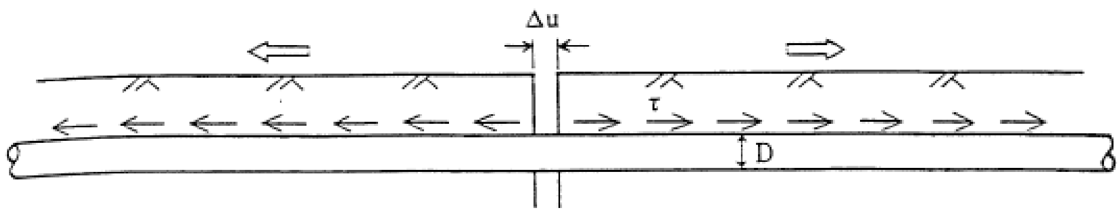
The vertical displacement is half of the horizontal. The guide outlines four load deformation conditions shown in Figure 5-1 and a Deformability Index is used to estimate pipe capacity. The Deformability Index includes strain capacity of the pipe and of the joint.

IEEE Standard 344-1987 - The IEEE has developed a standard for the seismic qualification of equipment for the nuclear industry.

Performance Objectives

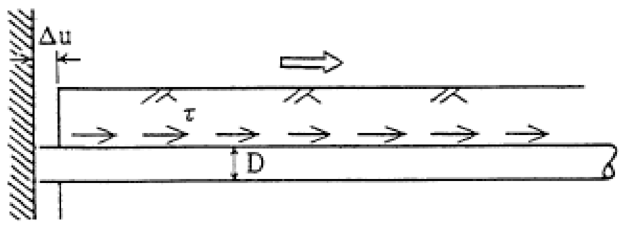
The development of performance objectives is the first step in development of a general criteria for lifelines. The following performance objectives are presented herein and represent a new synthesis proposed for use. They are based on mandates of public law and extensions of current criteria.

Ordinary Construction / Ordinary Lifelines - Lifeline service associated with construction categorized as "ordinary" shall be designed with the same levels of service. In general ordinary construction is expected to

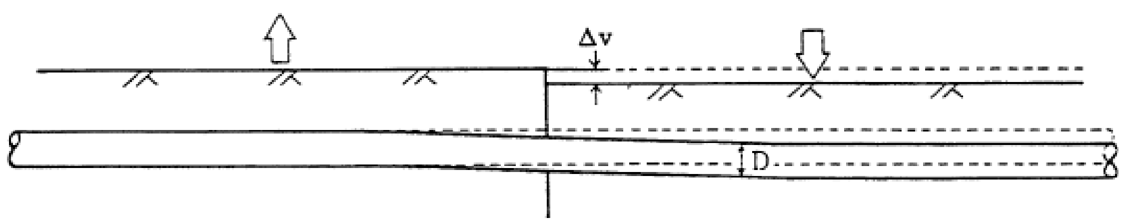


Δu : Allowable ground displacement = Deformability Index (Horizontal)
τ : Soil Restriction

a. Horizontal Relative Displacement between two Ground Blocks

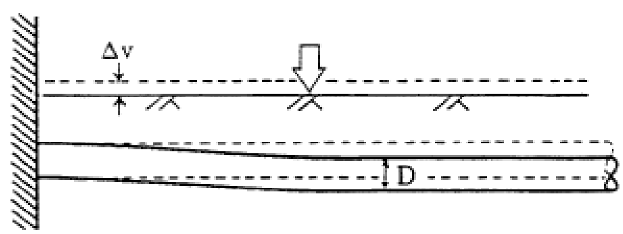


b. Horizontal Displacement in a Ground Block Adjacent to a Rigid Structure



Δv : Allowable ground displacement = Deformability Index (Vertical)

c. Vertical Relative Displacement between two Ground Blocks



d. Vertical Displacement in a Ground Block Adjacent to a Rigid Structure

Ground-Displacement Models for Deformabilitty Evaluation for Buried Pipelines

Figure 5-1. Ground-displacement models for pipe deformation.

- Resist a moderate level of ground motion without damage;
- Resist a major level of earthquake ground motion without collapse, but with structural as well as nonstructural damage.

Wharves and Piers Lifelines associated with pier or wharves shall be designed with the same levels of service.

- Resist a moderate level of ground motion without damage;
- Resist a major level of earthquake ground motion without collapse, and with the structural in a repairable condition.

Essential Construction / Essential Lifelines - Lifeline service associated with construction categorized as "essential" shall be designed with the same levels of service. In general essential construction is expected to:

- Resist the earthquake likely to occur one or more times during the life of the structure with minor damage without loss of function and the structural system to remain essentially linear.
  Resist the rare earthquake with a low probability of being exceeded during the life of the structure without failure and without loss of acceptable levels of functionality.

Hazardous Materials/Lifelines - Lifeline service associated with construction categorized as "containing hazardous materials" shall be designed with the same levels of service. In general hazardous material containment construction is expected to:

- Resist pollution and release of a major spill of hazardous materials for a very rare event

Seismic Loads

The second element of a general criteria for lifelines is the specification of seismic load level to establish the ground motion and lateral load forces to be applied in design. It is based on current criteria and an extension of existing mandates logically applied to analogous situations.

Design Earthquakes

The following criteria are based on current criteria and public law. The lifeline systems shall be designed to resist the loading produced as follows:

- **Ordinary category of construction on average seismicity sites**

For sites of average seismicity, use NEHRP provisions, which establishes the earthquake at a nominal 10 percent chance of exceedance in 50 years.

- **Pier or wharf category of construction**
  Sites where the lifeline is associated with a pier or wharf shall use a two-earthquake procedure with Level 1 and a Level 2 based on a local site seismicity study. Values less than NEHRP code are not be permitted

- **Essential category of construction**
  Sites where the lifeline is deemed important and essential shall use a two-earthquake procedure with Level 1 and a Level 3 earthquake based on a local site seismicity study. Values less than code are not be permitted.

- **Construction containing polluting or hazardous material**
  A Level 4 earthquake exposure shall be used.

In addition to seismic ground motion there are additional hazards which must be considered:

- Fault movement and ground displacement
- Liquefaction and associated lateral spreading, settlement flow slides, loss of support and buoyancy of buried tanks.
- Landslides
- Tsunamis

Modification to Design Ground Motion

Lifelines consist of a variety of elements some of which are substantial structures such as tanks, transformer stations and bridges, others are distributed elements such as buried pipelines, power lines and railroad tracks, and others are components within structures such as internal equipment, transformers, and other building elements. The ground motions used in design of lifelines may differ from the motions used in conventional building design since the seismic motion on the lifeline may be substantially different than that associated with free-field ground motion. For component elements located within a structure the lifeline component design motion can be substantially amplified by the response of the structure. In such cases the motion to be used for design of the component must be the local seismic motion transmitted by the structure to the component. The dynamic coupling between the component and the structure must be taken into account if the component is of a size sufficient to influence the response of the structure. Large differential motions may be produced on components which are supported at multiple locations.

Chapter 6 of NAVFAC P355.1 illustrates the procedure for calculation of the maximum floor accelerations using the linear response spectra technique. A modal participation factor is applied to the story modal acceleration response to determine the modal spectral acceleration to be applied to the lifeline component. A design response spectrum is constructed using the modal floor accelerations, the participation factors, a magnification factor, and the period of the lifeline component. Response spectra techniques have been utilized for at least the last 35 years. They offered a means for performing dynamic analysis more accurately than pseudostatic approaches. The response spectra technique is a linear procedure. A structure responding to a major earthquake is expected to sustain significant nonlinear behavior. The ability of response spectra techniques to accurately track displacements reduces as the amount of nonlinearity increases. With the evolution of the desktop computer, nonlinear finite element techniques which previously required extensive mainframe computer time, have now been developed which can offer a potentially more accurate analytical alternative.

Lifeline Performance During Recent Earthquakes

Understanding the behavior and possible failure mechanisms of a lifeline structure is important in the development of a design criteria for safe operation. Part of understanding the performance of a lifeline structure in an earthquakes involves understanding the design from which the structure was constructed. and the construction practice used in its erection. Werner and Hung (1982) gives an excellent compilation of case studies mostly recounting Japanese experiences from the 1920's to 1980. They conclude that "By far the most significant source of earthquake-induced damage to port and harbor facilities has been porewater pressure buildup... which has led to excessive lateral pressures applied to quay walls and bulkheads." They cite the 1964 Niiagata and 1964 Alaska earthquake where "porewater pressures buildup has resulted in complete destruction of entire port and harbor areas" They note that direct effects of earthquake induced vibrations on waterfront structures is minimal and overshadowed by liquefaction induced damage. In the 1978 Sendai earthquake a major oil refinery with 90 storage tanks had three fail and three damaged. Additionally a large welded steel plate tank pulled out of its concrete embedment. A summary of recent lifeline experiences during earthquakes follows.

Alaska Earthquake - The 1964 caused considerable damage to oil storage tanks by tsunamis, earth settlement, and liquefaction. Damage to Union Oil tanks in Whittier caused fires. In Anchorage seven tanks collapsed releasing combustible fluids; three additional Standard Oil tanks released 750,000 gallons of aviation fuel. This experience led to a change in tank design, Eguchi (1987)

San Fernando Earthquake- The 1971 San Fernando earthquake resulted in direct losses to the electric power systems of $33 million. It caused distress to numerous tanks. Bulging of the lower section of about 12-inches above the base was noted extensively and termed "Elephant's Footing". Ductile steel pipelines were able to withstand ground shaking but could not withstand ground deformation associated with faulting and lateral spread. Eleven transmission pipelines were damaged by liquefaction induced lateral spread and landslides. Eighty breaks occurred to the underground welded steel transmission pipeline located in the upper San Fernando Valley, the most serious in a 1930 old oxyacetylene-welded pipeline. Although located in an uplift zone the failure was caused by compressive forces wrinkling the pipe, Eguchi (1987). Newer pipelines in the same area did not fail. There were 18 documented hazardous material releases resulting in 6 fires. There was damage to the Jensen water treatment plant resulting in an outage. The absence of an inlet to outlet bypass was noted as a factor in impeding the problem of restoration of service.

Santa Barbara - In the 1978 magnitude 5.1 Santa Barbara earthquake a train derailed shortly after the earthquake from damaged tracks. About 40 cars derailed at a speed of 50 miles per hour.

Coalinga Earthquake - The 1983 Coalinga earthquake had adequately designed pipelines which remained serviceable; however large vertical tanks containing molasses tilted and in one case overturned. This was initiated by large deformations in the steel support frame. At a treatment plant, chlorine tanks on standard saddle supports slid up to 10 inches. The valves on a 1-inch line to a clarifying tank shook open causing a major oil spill. Anchors on a 12 kV transformer broke. A hazardous material spill resulted in significant damage to a high school; there were three other hazardous material incidents of significance. Numerous breaks in the natural gas line occurred but fires did not occur since the main valve was closed manually shortly after the earthquake. Several tank and pipeline failures occurred in oil drilling and processing facilities. In general it was noted that secondary containment systems functioned well. most pipe breaks occurred at pipe connections.

Whittier Narrows- The 1987 magnitude 6.1 earthquake demonstrated that well designed process pipelines can perform well. Damage where it occurred was usually limited to sections that were corroded or anchored at two locations which experienced large lateral relative displacement. A 1-ton relocatable gas cylinder being filled with chlorine started to roll down the loading platform breaking the connection causing a significant chlorine release. Southern California Gas reported 1411 gas leaks were directly caused by the earthquake. Portions of the California State University, Los Angeles were without gas for 12 weeks. Five fires were reported; three of these were attributed to gas leaks. There were about 30 hazmat calls for assistance.

Loma Prieta Earthquake- The 1989 magnitude 7.1 Loma Prieta Earthquake caused failure of many pipelines and tanks. The Port of Redwood City is located at the southern end of the San Francisco Bay. The Port contains tanks for petroleum. The Port was constructed on Bay Mud. Damage consisted primarily of broken water lines and damaged batter piles. The Port of Richmond is located at the northeast end of the San Francisco Bay and handles petroleum products and liquid bulk cargo. Portions of this port are constructed on rock and other portions on fill. The primary damage was the rupture of a gasoline storage tank at the UNOCAL terminal. Fuel was contained in the surrounding berm. Some liquefaction was reported in undeveloped areas of the port. Broken waterlines occurred at the Ford plant from liquefaction and excessive soil pressures. The Port of San Francisco is located on the west side of the San Francisco Bay and handles general cargo. The port is constructed on fill. The primary damage was liquefaction and settlement. Numerous buildings were damaged water and gas lines broke. The Port of Oakland is located on the east side of the San Francisco Bay on fill. The port sustained wharf damage and noted batter pile failures. Liquefaction of the fill produced settlement and lateral spread. Horizontal accelerations were measured at the wharf and ranged from about 0.3g to 0.45g. Cranes suffered damage and water lines broke. Fire lines ruptured eliminating fire fighting protection, Seed et al. (1990). Tank failure modes consisted of "elephant's foot " bulging, vertical splitting of tank wall, puncture of the tank wall by restrained pipe, pipe damage from differential anchorage motion. Hazardous material spills occurred in several industrial and a few commercial facilities. Over 300 liquid hazardous spills occurred in the San Francisco and Monterey Bay areas as a result of ruptured tanks, pipe leaks, equipment leaks, and broken containers. It appears that secondary containment was generally effective. At least 50 instances occurred of release of hazardous gases other than natural gas. There were 3 to 4 leaks on a high pressure gas main and between 300 to 400 leaks on low pressure gas lines.

The Navy sustained 44 pipeline breaks in pipes up to 16 inches in diameter on Treasure Island. They included 28 fire and freshwater lines of steel or asbestos cement, 10 sewage lines of vitrified clay and 6 welded-steel gas lines, Egan and Wang (1991). Many of the breaks occurred near the dike in areas of high lateral spreading. Crude estimates of lateral spreading required to cause failure are:

Type	Pipe Diameter	Spreading to Induce Failure
Steel or Asbestos Cement	1 to 4 in	1 inch
Steel or Asbestos Cement	12 to 16 in	6 to 12 inches
Vitrified clay pipe		1/4 inch

Soil liquefaction caused damage to the terminal facilities much of which were on filled land composed of loose dumped or hydraulically placed sand underlain by soft normally consolidated Bay Mud. Liquefaction of the fill resulted in settlements and lateral spreading, cracking the pavement over a wide area. Maximum settlements of the paved yard area were up to a 12 inches.

In the Monterey area water tanks belonging to PG&E were damaged and one ruptured apparently as a result of foundation softening and displacements. Settlements of several inches were noted and there were breaks in utility lines. Pile supported facilities were not damaged.

Transportation facilities sustained $1 billion in damage including $200 million to the Cypress Street elevated viaduct. Numerous roads were closed by pavement damage, landslides, or bridge damage. A 3000-foot section of runway was severely damaged having several breaks as large as 30 inches in width. Undulations were noted in the pavement along with settlement. The pavement was situated was on 10 to 15 feet of unconsolidated hydraulically dredged sand fill which experienced extensive liquefaction. The runway at the Naval Station, Alameda cracked and moved laterally from liquefaction of the soil below. The Port of Oakland experienced liquefaction damage to paved yard areas Batter piles in wharves were damaged.

Big Bear Earthquakes- On June 28, 1992 two earthquakes occurred in San Bernadino County, California, a magnitude 7.5 at 4:58 AM and a magnitude 6.6 at 8:04 AM. These two events were followed by numerous aftershocks. Horizontal fault rupture displacement associated with these event was from 5 to 9.5 feet. Most pipeline damage was associated with the rupture zone. At least 6 water tanks ranging in size from 42,000 to 417,000 gallons were damaged. Damage consisted of elephant's foot bulging at the base, shell and roof damage, shell splitting at access hatches and broken pipe entering the tanks.

Guam Earthquake - On August 8, 1993 a magnitude 8.1 earthquake occurred 50 miles offshore and caused over $125 million in damages to Naval facilities on Guam. Nearly all of Guam is firm soil or rock except for the region containing the commercial and Navy ports which is composed of natural alluvium and artificial fill. It is estimated the peak horizontal ground accelerations were about 0.25g. Liquefaction was a major problem and lateral spreading of 1 to 2 feet was observed at wharf areas. It also resulted in settlements, backfill collapse and bulkhead movements. Buried water and power lines were fractured. Sheet piles failed in shear and deadman anchors pulled out. Pier batter piles failed in shear at the pile cap. Other Navy damage consisted of fuel tank leaks, sloughing of a dam, damage to masonry housing units and major damage to the power plant which supplied 20 percent of the islands power capacity.

Northridge Earthquake- On January 17 1994 a moment magnitude 6.7 earthquake occurred in Northridge. This event caused about 1,400 water, gas and fuel pipeline breaks in the San Fernando Valley area. Many of the breaks occurred in mapped areas of high liquefaction potential. Outside the zone of high liquefaction potential, the dispersed pattern of breaks is attributed to old brittle pipes damaged by ground movement. While much of the pipe damage is within the liquefaction zone, this did not correlate to areas of high structural damage in that a large amount of structural damage occurred outside the zone of high liquefaction potential. In the Granada Hills area pipe breaks from water mains resulted in soil erosion and formation of large craters. On Balboa Boulevard a 22-inch pipe suffered two breaks, one in tensile failure and the other in compressive failure. These pipe failures were located in a ground rupture zone perpendicular to the pipeline. Leaking gas ignited at several locations. Some broken water and gas lines were found to have experienced 6 to 12 inches of separation in extension. The area experienced widespread ground cracking and differential settlements. Liquefaction was not evident on the surface and may have occurred at depth leading to subsurface soil block movement.

Some of the surface cracking was associated with underlying bedrock movements associated with primary or secondary faulting. A 85 inch sewage pipe ruptured in the Jensen Filtration Plant and a large reservoir settled 2 to 4 inches. The San Fernando Power Plant Tailrace, a 600 by 110 foot asphalt lined pond was breached. Lateral spreading was noted. A water storage tank east of Highway 5 at Valencia Boulevard collapsed. The Port of Los Angeles sustained peak horizontal accelerations on the order of 0.1 to 0.2g which resulted in liquefaction of hydraulic fill damaging crane rails, disruption of utilities, ground cracking and lateral spreading of up to 6 inches. All of the damage was of a relatively minor nature.

Kobe Earthquake- On January 17 1995, the Hyogo-ken Nambu (Great Hanshin Kobe) earthquake, Japanese magnitude 7.2 (about 6.9 moment magnitude), occurred in Kobe Japan. This event produced major damage to Japan's second busiest port, Matso (1995). Liquefaction was a major contributor to the extent of the damage producing typical subsidence of a half meter. Piles were used extensively in this area. They were designed to account for the negative skin friction and additional ground improvement was also performed. Structures on such piles performed well even though major subsidence occurred in surrounding areas. Other structures not on piles suffered differential settlement and tilting and significant damage. Liquefaction caused up to 3 meters of lateral spread displacement, sunk quay walls, broke utility lines, and shut down 179 out of 186 berths at the port. Numerous tank failures were reported, mostly caused by uplift of unanchored tanks. One LNG tank cracked requiring the evacuation of 80,000 people. Six well-braced large spherical tanks sustained no damage. Liquefaction was responsible for major damage to crane foundations. Hydraulic fill behind concrete caisson perimeter walls fill liquefied causing the caissons to move outward, rotating up to 3 degrees, and settling from 0.7 to 3.0 meters. The caissons were designed for a lateral coefficient of 0.1g. A seismic coefficient of 0.2g was usually used in the design of dockside cranes. Peak accelerations of 0.8g in the NS direction, 0.6g in the EW direction and 0.3g vertical were noted from accelerograph recordings. The event had a duration of about 20 seconds. Most damage is attributed to liquefaction of backfill and associated pressures and settlements and lateral deformations since structures supported on piles suffered much less damage, Liftech (1995). It should be noted that caissons designed for 0.25g sustained lower levels of damage.

Liquefaction And Lifelines

Design of structures shall include provisions to evaluate and resist liquefaction of the foundation and account for expected potential settlements and lateral spread deformation. Special care will be given to buried pipelines in areas subject to liquefaction to preclude breaks resulting in release of a major spill of hazardous materials. The most important element in seismic design of pipelines is proper siting. It is imperative to avoid areas of landslide and lateral spread.

The presence of any potentially liquefiable materials in foundation or backfill areas shall be fully analyzed and expected settlements computed. Specific attention shall be paid to the acceptability of the amount of settlements. Since liquefaction is a major damage mechanism at the waterfront, remediation is a mandatory requirement where the risk of a pipeline break or tank failure is shown by computation to be possible and hazardous materials would be expected to be released.

Water, Gas And Liquid Fuel Lifelines

Pipelines

Pipelines must be designed to resist the expected earthquake induced deformation state and induced stresses. It is common practice to design the pipe support or embedment based on the nature of the soil encountered. Where marginal soil is encountered, pipelines can be supported on piles above ground or placed within larger pipes to allow ground movement. Generally permissible tensile strains are on the order of 1 to 2 percent for modern steel pipe. This is based on observation that steel pipelines have been observed not to rupture at tensile strains between 2 to 5 percent. Higher local strains have been noted. Pipelines have experienced wrinkling in compression at strains much less than the tensile limits; however this does not of itself constitute failure. A rule of thumb states that the onset of wrinkling occurs at strains of about 0.3 times the ratio of wall thickness to radius. Welded steel pipes have performed well during earthquakes. The quality of weld is very important. There appears to be more failures with oxyacetylene-welded steel pipes compared to arc-welded steel pipes. The difference may not be the type of weld but may be the weld quality. Corrosion of pipelines reduces their ability to withstand seismic forces. Pipeline damage seems inversely proportional to pipe diameter caused by an increase in stiffness with larger size pipe which makes it more able to resist deformation. Expressed in another form, pipeline strength is proportional to diameter. An exception to this seems to be steel pipe with a lap welded joint where strength decreased with increasing diameter. Also gasketed joints seem to be 5 or more times more likely to fail than welded joints. In addition to tensile and compressive failures, buckling failures are possible. The presence of bends, elbows and local eccentricities tends to concentrate deformation at these locations.

To accommodate differential motion between pipelines and storage tanks it is recommended that a length of pipeline greater than 15 pipe diameters extend radially from the tank before allowing bends and anchorage and that subsequent segments be of length not less than 15 diameters. Flexible couplings should be used on long pipelines. In general pipes should not be fastened to differentially moving components; rather, a pipe should move with the support structure without additional stress. Unbraced systems are subject to unpredictable sway whose amplitude is based on the system fundamental frequency, damping and amplitude of excitation. For piping internal to a structure, bracing should be used for system components. Flexible grooved pipe couplings can reduce the transmission of stresses and resilient gaskets can dampen vibration.

Manufacturers specification give guidance on linear and angular movement tolerances. "Grooved-end mechanical pipe couplings do not simultaneously provide maximum linear and angular movement. However, systems designed with enough joints, thus allowing for recommended tolerances, will accommodate both", Greene (1993). When large movements are anticipated seismic swing joints composed of flexible couplings, elbows and nipples can be used. Provisions for expansion must be included.

Machinery and pumps are often acoustically isolated by use of loose connections to minimize vibration transmission such as by use of slotted holes. Snubbers by definition are restraints with an air gap. Such anchorages can amplify seismic motion by having equipment bang against restraints. Use of resilient grommets or molded epoxy grouting can eliminate the air gap and avoid hard surface contact. The snubber and the connection of the snubber to the equipment and structure must have sufficient strength to transmit the inertial forces. The Northridge earthquake has shown that use of rails is not a satisfactory method of restraint and such usage was associated with many failures of welds and dislocation from the rails. Suspended pipelines can also resonate with the earthquake if not sufficiently restrained. Sway of suspended components must be restrained. Seismic isolation can be an effective technique for reducing loading on floor mounted equipment. Seismic isolation can be used in addition to snubbers or can be made a part of the snubber. While there are no standards for seismic snubbers, their capacities should be stated by the manufacturer and a rating is assigned by The Office of Statewide Health Planning and Development of the State of California. Proper anchorage capacity including both horizontal shear and overturning uplift is required and a wedge anchor is recommended. Poured in place anchors are not feasible for snubber tie-down since equipment location is variable and often not defined specifically. Snubbers must be omnidirectional with at least a 3/8 inch resilient collar at least 4 snubbers must be used and all snubbers must be rated, Lama (1994).

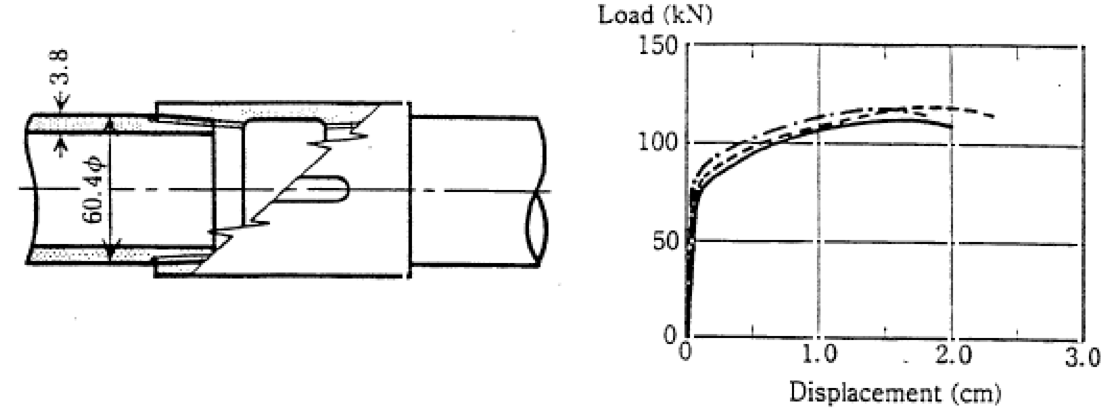
Nishio (1992) presents information of pipeline design in Japan. Figures 5-2 and 5-3 show pipe joint capacities for several Japanese pipe couplings. This excellent reference illustrates how the Japanese Gas Association provisions were developed and provides example calculations of their Deformability Index. The paper presents a discussion of the provisions and notes that the provisions use a value of deformation of 5.0 cm independent the liquefaction potential. Nishio introduces a probabilistic basis for assessing damage based on sample size. He shows the deformation capacity increases with diameter of the pipe.

In the analysis of continuous pipelines, it is possible to estimate the axial strain of the pipe in terms of the maximum ground strain:

$$\varepsilon_{p,\,max} = V_{max} \;/\; c_p$$

and the maximum curvature of the pipeline

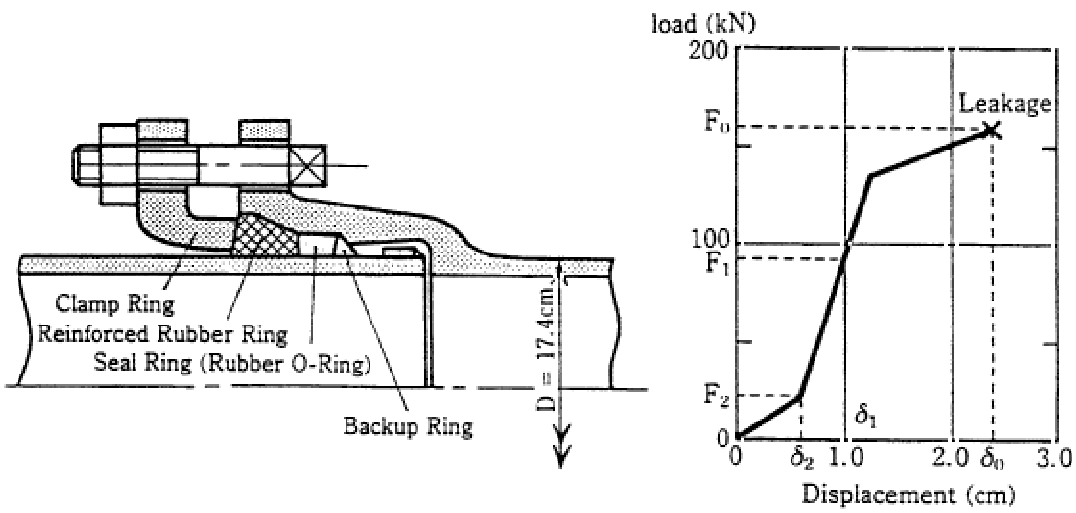
$$\chi_{p,max} = A_{max} \;/\; c_s^2$$



a. Cross Section of Thread-Joint (Socket, 50 mm)

b. Load-Displacement Diagram of 50 mm Thread-Joint (Axial Tension)

Cross Section and Load-Displacement Diagram of 50 mm Thread Joint

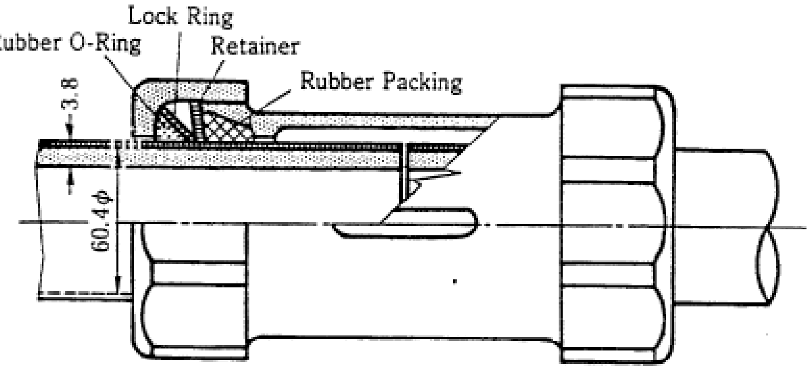


a. Cross Section of TM Joint

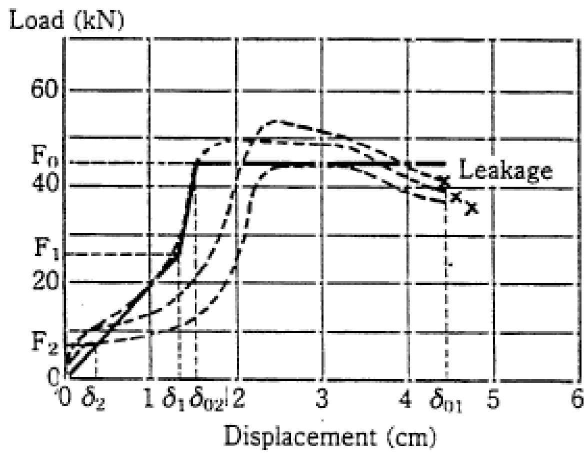
b. Load-Displacement Diagram for 150 mm TM Joint

Cross Section and Load-Displacement Diagram of Tokyo-Gas Type Mechanical Joint (TM Joint) for Ductile Cast-Iron Pipe (150 mm)

Figure 5-2. Cross section of Japanese pipe joints and strength.



a. Cross Section of Mechanical Joint for Steel Pipe (Tokyo Gas LM-Type Socket, 50 mm)



b. Load-Displacement Diagram of 50 mm Mechanical Socket

----- Experiments

——— Diagram for Calculation

Cross Section and Load-Displacement Diagram of Mechanical Joint for Steel Pipe (50 mm Socket)

Figure 5-3. Cross section of Japanese pipe joint and strength.

where

A_{max}	Maximum ground acceleration
C_p	Compression wave propagation velocity
C_s	Shear wave propagation velocity
V_{max}	Maximum ground velocity

The maximum pipe joint displacement and joint rotation can be estimated by:

$$U_p = \varepsilon_{p,\, max}\, L$$

$$\theta_p = \chi_{p,max}\ L$$

where

L Length of pipe segment

Note that C_s and C_p can be estimated from G and as follows:

$$C_s = \sqrt{\frac{G}{\rho}} \quad \text{and} \quad C_p = \sqrt{3C_s}$$

Eguchi et al. (1994) present an analysis of lifeline system damage which gives a good insight into the performance of pipelines. Table 5-1 presents relative performance of various types of pipe to shaking, liquefaction, landslide and fault rupture. They have compiled data on the number of repairs per 1000 feet of pipe and developed Figure 5-4 for fault rupture and ground shaking. The symbols are identified in Table 5-1. They note that the two mechanisms of ground displacement/fault rupture and shaking are different with the former being more damaging. Figure 5-5 shows their estimate of relative pipe performance under liquefaction and landslides conditions. Pipe diameter while a factor in pipe performance it was found that pipe material and joint type were more significant factors in normalizing field data. The data is intended to give system relative performance and not to be used to evaluate a single pipe Wang et al. (1992) illustrate use of flexible pipe joints, Figure 5-6.

The provisions of NAVFAC P355 Chapter 12 Section 12-7d pertain to design of essential pipelines and are part of this specification. They are as follows:

d. Seismic restraint provisions. Seismic restraints that are required for piping ..... will be designed in accordance with the following provisions.

(1) *General* The provisions of this paragraph apply to the following: ....

Table 5-1

RELATIVE PIPELINE VULNERABILITIES

Pipe Type	Ground Shaking (MMI)		Liquefaction, Lurching	Landsliding	Fault Rupture Displacement, (Inches)	
	8	10			10	100
WSAWJ (Modern)	L	L	L	L	L	L
WSAWJ (Pre-WWII)	L	L	M	M	M	M
WSGWJ (Pre-WWII)	M	H	H	H	H	H
WSCJ	M	M	H	H	H	H
CI	M	M	H	H	H	H
AC	M	M	H	H	H	H

Source: Eguchi (1983)

Pipe Types:

WSAWJ = Welded Steel Pipe with Arc-Welded Joints
WSGWJ = Welded Steel Pipe with Gas-Welded Joints
WSCJ = Welded Steel Pipe with Caulked Joints
CI = Cast Iron
AC = Asbestos Cement

Vulnerability Levels:

Low = No pipeline leaks expected.
Moderate = Some repairs expected.
High = Likely to fail or require repair given this level of seismic intensity or effect.

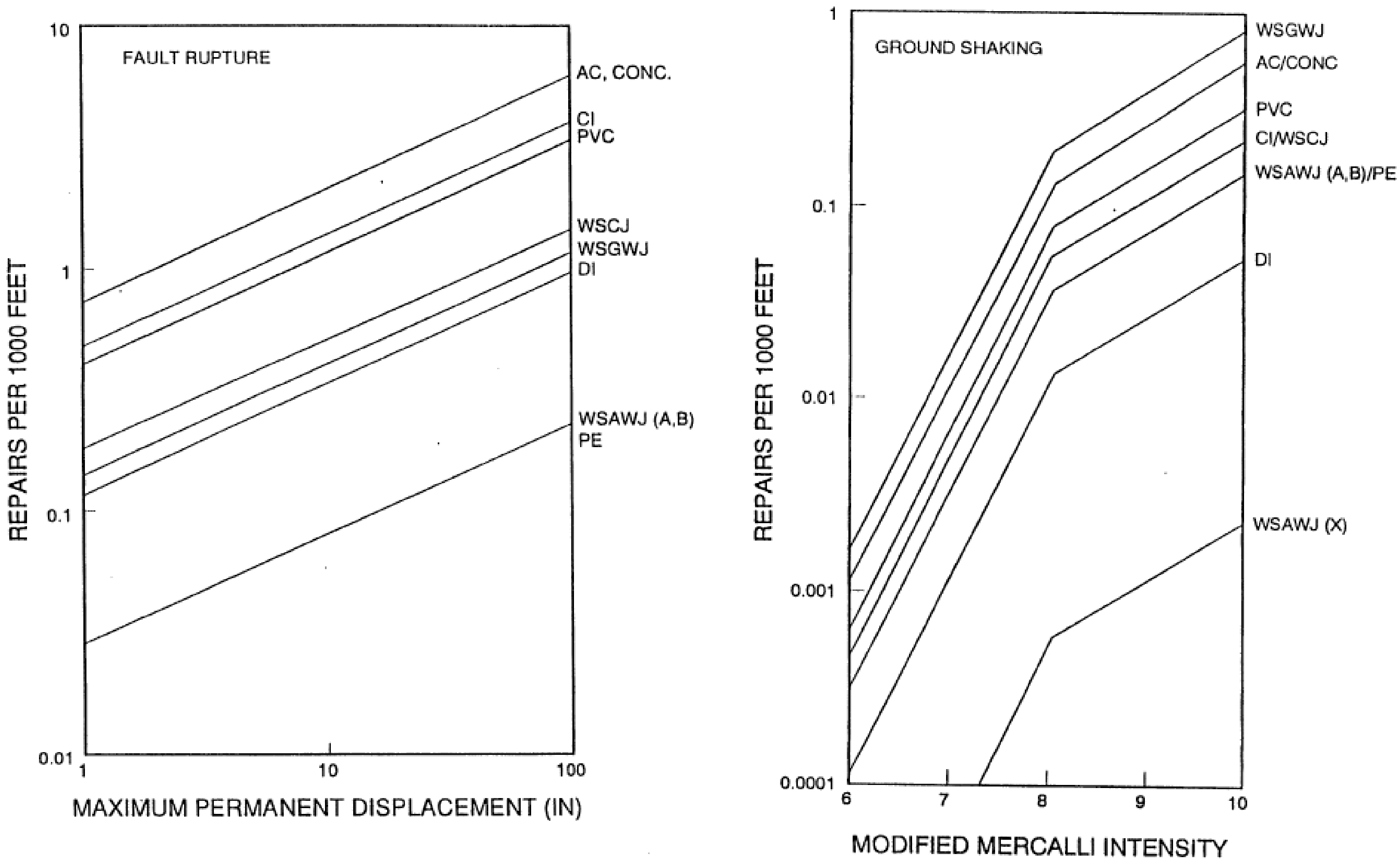


Figure 5-4. Pipe repair model for fault rupture and shaking, from Eguchi et al. (1994).

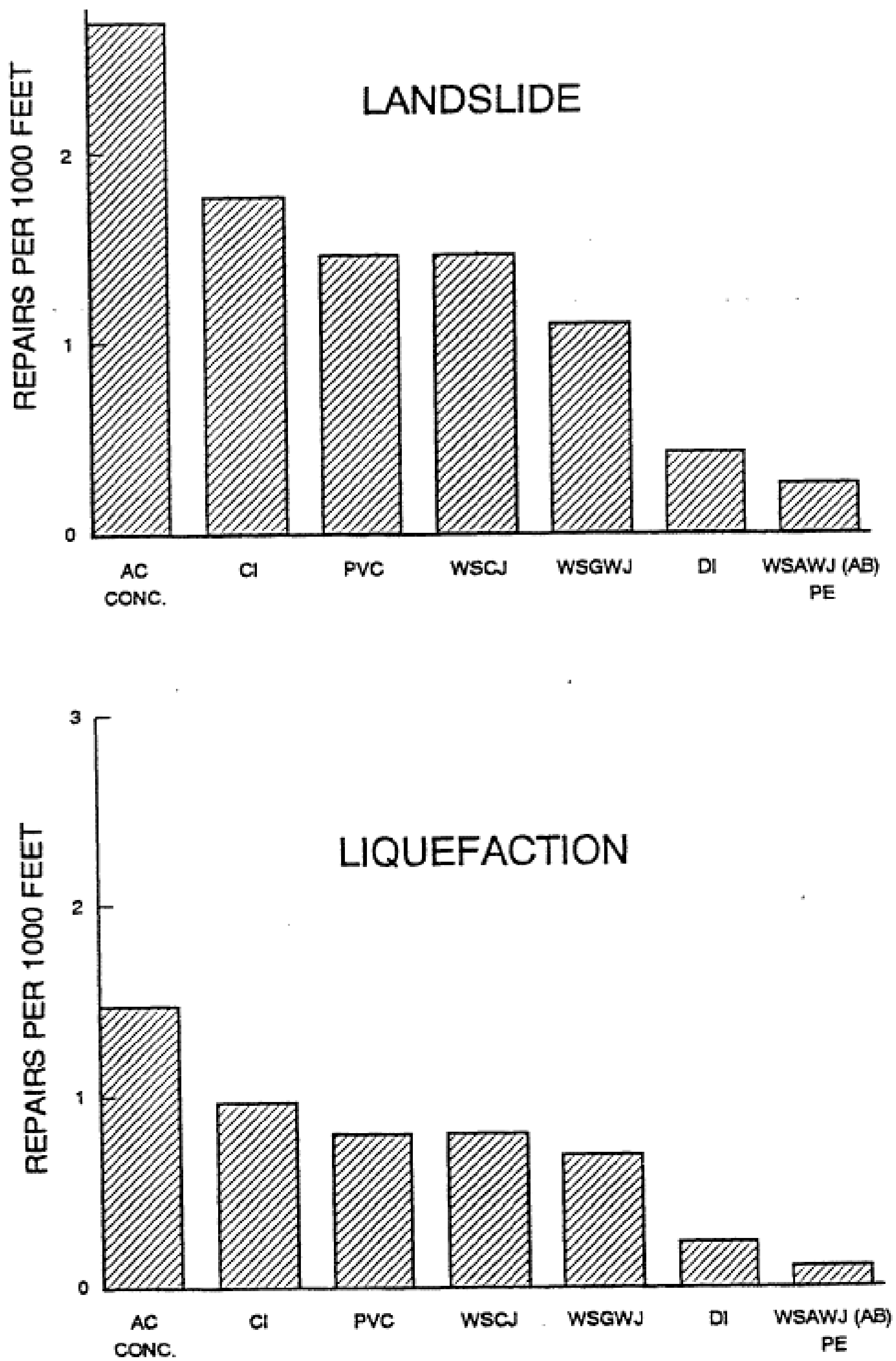
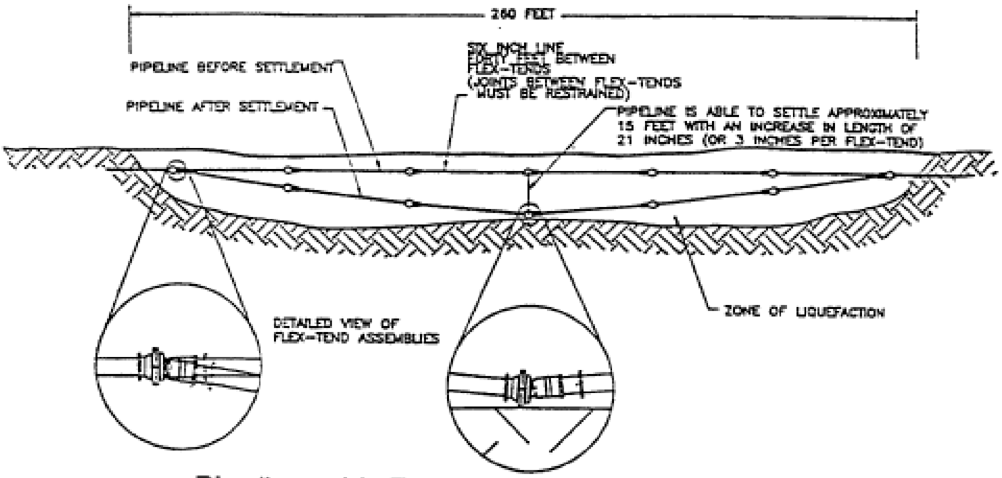
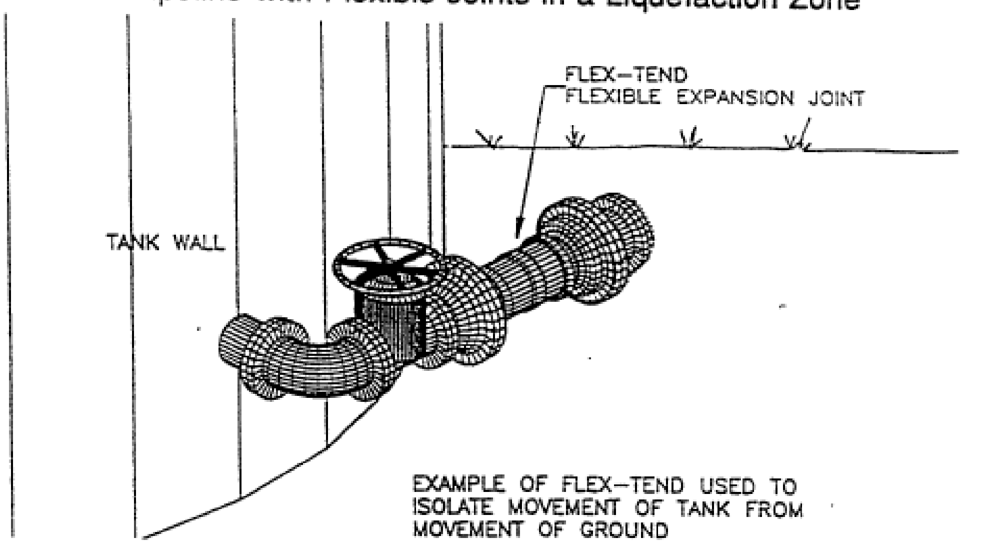


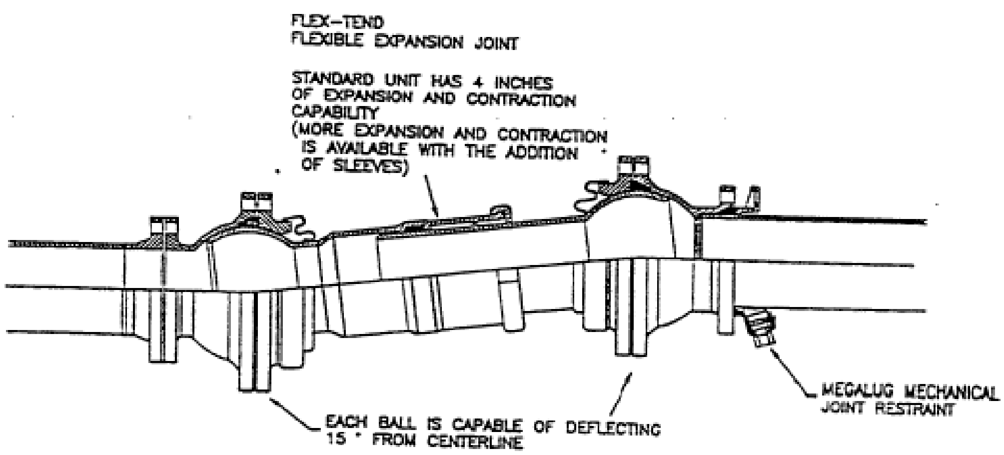
Figure 5-5. Pipe repair model for landslide and liquefaction, from Eguchi et al. (1994).



Pipeline with Flexible Joints in a Liquefaction Zone



Flexible Joint for Pipe-Structure Connection



A Flexible Joint Cross Section

Figure 5-6. Flexible pipe connections.

(b) Horizontal pipe. All horizontal pipes and attached valves. For the seismic analysis of horizontal pipes, the equivalent static force will be considered to act concurrently with the full dead load of the pipe, including contents.

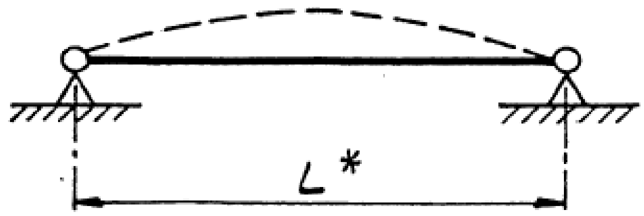
(c) Connections. All connections and brackets for pipe will be designed to resist concurrent dead and equivalent static forces. The seismic forces will be determined from the appropriate provisions below. Supports will be provided at all pipe joints unless continuity is maintained. See paragraph (4) below for acceptable sway bracing details.

(d) Flexible couplings and expansion joints. Flexible couplings will be provided at the bottoms of risers for pipes larger than 3½ inches in diameter. Flexible couplings and expansion joints will be braced laterally unless such lateral bracing will interfere with the action of the flexible coupling or expansion joint. When pipes enter buildings, flexible couplings will be provided to allow for relative movement between soil and building.

(e) Spreaders. Spreaders will be provided at appropriate intervals to separate adjacent pipe lines unless the pipe spans and the clear distance between pipes are sufficient to prevent contact between the pipes during an earthquake.

(2) *Rigid and rigidly attached piping Systems.* Rigid and rigidly attached pipes will be designed in accordance with paragraph 12-3. The equivalent static lateral force is given by $F_p = ZI_pC_pW_p$ (SEAOC eq 1-10), where C_p is equal to 0.75 and is the weight of the pipes, the contents of the pipes, and the attachments. The forces will be distributed in proportion to the weight of the pipes, contents, and attachments. A piping system is assumed rigid if the maximum period of vibration is 0.05 second (for pipes that are not rigid see paragraph (3) below). Figures 12-4, 12-5, and 12-6, (Shown in this report as Figures 5-7, 8 and 9) which are based on water-filled pipes with periods equal to 0.05 second, are to be used to determine the allowable span-diameter relationship for Zones 1, 2, 3, and 4 for standard (40S) pipe; extra strong (80S) pipe; Types K, L, and M copper tubing; and 85 red brass or SPS copper pipe.

(3) *Flexible piping Systems.* Piping systems that are not in accordance with the rigidity requirements of paragraph 12-7c(2) (i.e., period less than 0.05 second) will be considered to be flexible (i.e., period greater than 0.05 second). Flexible piping systems will be designed for seismic forces with consideration given to both the dynamic properties of the piping system and the building or structure in which it is placed. In lieu of a more detailed analysis, the equivalent static lateral force is given by $F = ZI_pA_pC_pW_p$ (eq 12-2), where $A_p = 5.0$, $C = 0.75$, and is the weight of the pipes, the contents of the pipes, and the attachments. The forces will



DIAMETER INCHES	STD. WT. STEEL PIPE 40 S	EX. STRONG STEEL PIPE 80 S	COPPER TUBE TYPE K	COPPER TUBE TYPE L	COPPER TUBE TYPE M	85 RED BRASS & SPS COPPER PIPE
1	6'-6"	6'-6"	5'-0"	4'-9"	4'-6"	5'-6"
1½	7'-6"	7'-9"	5'-9"	5'-6"	5'-6"	6'-6"
2	8'-6"	8'-6"	6'-6"	6'-6"	6'-3"	7'-0"
2½	9'-3"	9'-6"	7'-3"	7'-0"	7'-0"	8'-0"
3	10'-3"	10'-6"	7'-9"	7'-6"	7'-6"	8'-9"
3½	11'-0"	11'-0"	8'-3"	8'-3"	8'-0"	9'-3"
4	11'-6"	11'-9"	9'-0"	8'-9"	8'-6"	9'-9"
5	12'-9"	13'-0"	10'-0"	9'-6"	9'-6"	10'-9"
6	13'-9"	14'-0"	10'-9"	10'-6"	10'-3"	11'-6"
8	15'-6"	16'-0"				
10	17'-0"	17'-6"				
12	18'-3"	19'-0"				

*MAXIMUM UNSUPPORTED OR UNBRACED LENGTHS (L) ARE BASED ON WATER-FILLED PIPES WITH PERIOD (T_a) EQUAL TO 0.05 SEC. WHERE

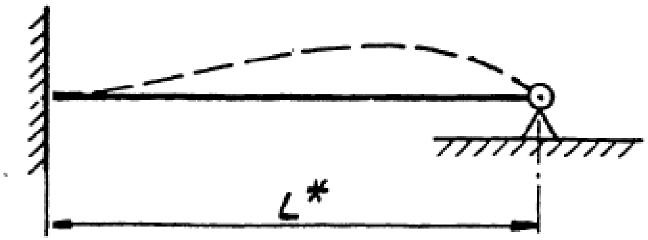
$L^2 = 0.50\pi\, T_a \sqrt{EIg/w}$

E = MODULUS OF ELASTICITY OF PIPE

I = MOMENT OF INERTIA OF PIPE

w = WEIGHT PER UNIT LENGTH OF PIPE AND WATER

Figure 5-7. Maximum Span for rigid pipe, pinned-pinned.
From NAVFAC P355 Figure 12-4

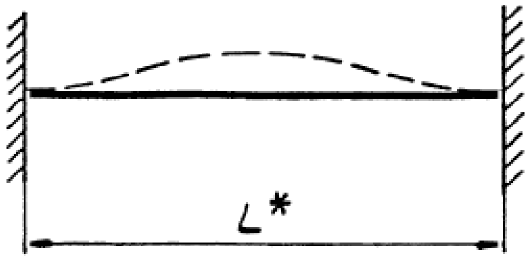


DIAMETER INCHES	STD. WT. STEEL PIPE 40 S	EX. STRONG STEEL PIPE 80 S	COPPER TUBE TYPE K	COPPER TUBE TYPE L	COPPER TUBE TYPE M	85 RED BRASS & SPS COPPER PIPE
1	8'-0"	8'-0"	6'-0"	6'-0"	5'-9"	6'-9"
1½	9'-6"	9'-6"	7'-3"	7'-0"	7'-0"	8'-0"
2	10'-6"	10'-9"	8'-0"	8'-0"	8'-9"	9'-0"
2½	11'-9"	11'-9"	9'-0"	8'-9"	8'-6"	9'-9"
3	12'-9"	13'-0"	9'-9"	9'-6"	9'-3"	10'-9"
3½	13'-6"	14'-0"	10'-6"	10'-3"	10'-0"	11'-6"
4	14'-6"	14'-9"	11'-0"	11'-0"	10'-9"	12'-3"
5	16'-0"	16'-3"	12'-3"	12'-0"	11'-9"	13'-3"
6	17'-0"	17'-9"	13'-6"	13'-0"	12'-9"	14'-3"
8	19'-3"	20'-0"				
10	21'-3"	22'-0"				
12	23'-0"	23'-6"				

* MAXIMUM UNSUPPORTED OR UNBRACED LENGTHS (L) ARE BASED ON WATER-FILLED PIPES WITH PERIOD (T_a) EQUAL TO 0.05 SEC. WHERE

$$L^2 = 0.78\pi T \sqrt{EIg/w}$$

Figure 5-8. Maximum Span for rigid pipe, fixed-pinned. From NAVFAC P355 Figure 12-5



DIAMETER INCHES	STD. WT. STEEL PIPE 40S	EX. STRONG STEEL PIPE 80S	COPPER TUBE TYPE K	COPPER TUBE TYPE L	COPPER TUBE TYPE M	85 RED BRASS & SPS COPPER PIPE
1	9'-6"	9'-6"	7'-3"	7'-3"	7'-0"	8'-0"
1½	11'-6"	11'-6"	8'-6"	8'-6"	8'-3"	9'-9"
2	12'-9"	13'-0"	9'-9"	9'-6"	9'-6"	10'-9"
2½	14'-0"	14'-3"	10'-9"	10'-6"	10'-6"	11'-9"
3	15'-6"	15'-9"	11'-9"	11'-6"	11'-3"	13'-0"
3½	16'-6"	16'-9"	12'-6"	12'-3"	12'-0"	14'-0"
4	17'-3"	17'-9"	13'-6"	13'-0"	13'-0"	14'-9"
5	19'-0"	19'-6"	15'-0"	14'-6"	14'-3"	16'-0"
6	20'-9"	21'-3"	16'-3"	15'-9"	15'-6"	17'-3"
8	23'-3"	24'-3"				
10	25'-9"	26'-6"				
12	27'-6"	28'-6"				

* MAXIMUM UNSUPPORTED OR UNBRACED LENGTHS (L) ARE BASED ON WATER-FILLED PIPES WITH PERIOD (T_a) EQUAL TO 0.05 SEC. WHERE

$$L^2 = 1.125\pi T_a \sqrt{EI g/\omega}$$

Figure 5-9. Maximum Span for rigid pipe, fixed-fixed. From NAVFAC P355 Figure 12-6

Diameter (in.)	Std. Wgt. Steel Pipe - 40S		Ex. Strong Steel Pipe - 80S		Copper Tube Type L	
	L*(ft)	F†(lbs)	L*(ft)	F†(lbs)	L*(ft)	F†(lbs)
1	22	70	22	80	11	17
1-1/2	25	140	26	180	12	35
2	29	220	30	290	14	70
2-1/2	32	380	33	460	15	110
3	34	550	35	710	17	150
3-1/2	36	730	38	930	18	220
4	39	960	40	1,200	19	300
5	41	1,440	44	1,900	20	470
6	45	2,120	46	2,750	22	730
8	49	3,740	54	5,150	26	1,550
10	54	6,080	59	7,670	28	2,620
12	58	8,560	61	10,350	31	3,950

*Maximum spans (L) between lateral supports of flexible piping are based on the resultant of an assumed loading of 1.5 w($ZI_pA_pC_p$ = 1.5) in the horizontal direction and 1.0 w (gravity) in the vertical direction. The resultant is 1.8 w.

The assumed maximum stress is 20,000 p.s.i. for steel and 7,000 p.s.i. for copper. Simple spans (pinned-pinned) are assumed. The calculated maximum lateral displacements are 3.5 inches for steel ($E = 29 \times 10^6$ p.s.i.) and 0.6 inch for copper ($E = 15 \times 10^6$ p.s.i.).

†The horizontal force (F) on the brace is based on 1.5 w L for the maximum span. For shorter spans, $F_{design} = (L_{design}/L)F$.

Figure 12-7. Maximum span for flexible pipes in Seismic Zone 4.

Zone	L (feet)	F (pounds)	$ZI_pA_pC_p$
3	1.1	0.8	1.12
2B	1.20	0.6	0.75
2A	1.25	0.5	0.56
1	1.35	0.3	0.28

Table 12-2. Multiplication factors for figure 12-7 for Seismic Zones 1, 2, and 3 or for cases where $ZI_pA_pC_p$ is not equal to 1.5.

Figure 5-10. NAVFAC P355 Figure 12-7 and Table 12-2.

be distributed in proportion to the weight of the pipes, contents, and attachments. Figure 12-7 (Shown in this report as Figure 5-10) may be used to determine maximum spans between lateral supports for flexible piping systems. The values are based on Zone 4 water-filled pipes with no attachments. If the weight of the attachments is greater than 10 percent of the weight of the pipe, the attachments will be separately braced, or substantiating calculations will be required. Temperature stresses have not been considered in Figure 12-7 (Figure 5-10 herein). If temperature stresses are appreciable, substantiating calculations will be required.

(a) *Use of Figure 12-7.* The maximum spans and design forces were developed for $ZI_pA_pC_p = 1.50$. For lower $ZI_pA_pC_p$ values, the spans and forces may be adjusted by the values in Table 12-2. (Figure 12-7 and Table 12-2 are reproduced in this report as Figure 5-10)

(b) Separation between pipes. Separation will be a minimum of four times the calculated maximum displacement due to F_p, but *not less* than 4 inches clear between parallel pipes, unless spreaders are provided ...).

(c) Clearance. Clearance from walls or rigid elements will be a minimum of three times the calculated displacement due to F_p, but not less than 3 inches clear from rigid elements.

(4) Alternative method for flexible piping systems. If the provisions in the above paragraphs appear to be too severe for an economical design, alternative methods based on the rationale described in paragraph 12-4 and paragraph 12-8 may be applied to flexible piping systems.

Figure 5-11 shows acceptable details for sway bracing from NAVFAC P355.

NAVFAC P355 Chapter 14 has several figures which illustrate good engineering practice for pipelines. Figure 14-1 from NAVFAC P355 shows a sewer manhole in which the bell is located at the manhole and encased in concrete to increase its strength while still providing flexibility to the mating pipe. When a pipeline passes through a wall good practice allows a 2 foot square space in the wall around the pipe; the space is filled with oakum or other expandable material to provide for differential movement. Good practice provides flexible couplings at both ends of a 90 degree bend and on each of the three sides of a tee connection. The manual suggests that prudent planning take into account the possible loss of electrical power to pumps and the potential need for manual operation of fuel pumps and backup lighting during an emergency. A properly designed pipeline distribution system will include alternative routes and valves to isolate potential pipe breaks and maintain operation with improved reliability.

The following are taken from Chapter 14 and are required in these criteria:

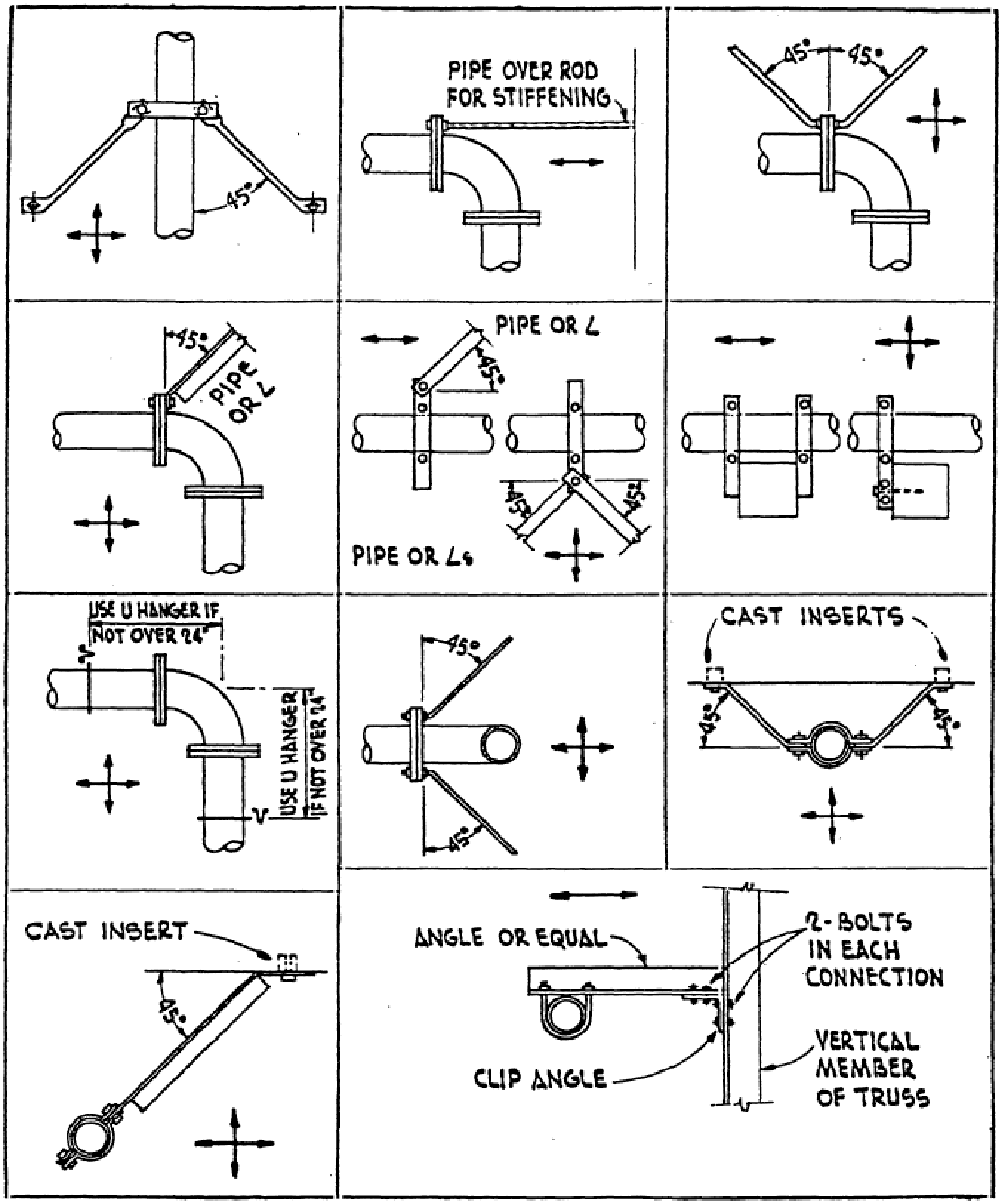


Figure 5- 11. Acceptable seismic details for sway bracing from NAVFAC P355 Figure 12-8.

No section of pipe in Zone 2, 3 or 4 shall be held fixed while an adjoining section is free to move, without provisions being made to relieve strains resulting from differential movement unless the pipe is shown to have sufficient stress capacity.

When secondary or standby gas supply systems cannot be justified for a site, gas distribution networks for buildings in Zones 2, 3 and 4 housing essential functions dependent upon gas shall include an above ground valved and capped stub. Provision will be made for attachment of a portable, commercial-sized gas cylinder system to this stub

For essential facilities in Seismic Zones 3 and 4, an earthquake activated gas shutoff valve shall be provided. If an earthquake activated shut-off valve presents the possibility of disrupted service in the buildings where the fire hazard is small, manually operated valves shall be installed.

Buildings housing essential functions shall be provided with two or more water service lines connected to separate sections of the supply grid to minimize loss of service. Service shall be interconnected within the building by check valves to prevent backflow.

Flexible connections shall be used between valves and lines for valve installation on pipes 3 inches or larger in diameter.

Flexibility shall be provided by use of flexible joints or couplings on a buried pipe passing through different soils with widely different degrees of consolidation immediately adjacent to both sides of the surface separating the different soils.

Flexibility shall be provided by use of flexible joints or couplings at all points that can be considered to act as anchors and at all points of abrupt change in direction and at all tees.

NAVFAC P355 paragraph 12.-7 (cited above) specifies restraints for critical piping in essential facilities.

Piping containing hazardous materials shall contain numerous valves and check valves to minimize release of materials if there is a break. A secondary containment system should be incorporated where feasible. When piping is connected to equipment or tanks, use of braided flexible hoses is preferable to bellow-type flexible connectors since the latter has been noted to fail from metal fatigue. Welded joints are preferable to threaded or flanged joints. If flanged joints can not be avoided the use of self-energizing or spiral wound gaskets can allow a bolt to relax wile continuing to provide a seal, Association of Bay Area Governments (1990). Seismic shutoff valves should be used where necessary to control a system or process. These systems can be triggered by a mechanical sensor on the valve or by a remote electronic sensor which can control a number of elements. Choice of valves should be restricted to approved valves to reduce leakage after closure.

New Tanks

Understanding the individual failure modes of individual tank components such as piping, restraints, tie down anchors, piles etc. is an important part of the development of a comprehensive design specification for quality performance. In general there are several types of tanks in use. Flat bottom vertical tanks vary in size from small 10,000 gallons to over several million gallons. Tanks can be anchored or unanchored. Large tanks can have internal columns to support the roof. Vertical tanks can be placed on a prepared mat or a ring foundation along the tank perimeter with or without edge confinement. Horizontal tanks are of cylindrical shape and are usually supported on two saddles. The typically range in size from 100 gallons to 10,000 gallons. Smaller tanks can be supported on legs. Typically fuel tanks for portable generators are of this type. Summers (1997) lists major causes of tank failure and includes the following:

- Buckling of the tank wall (termed elephant foot buckling)
- Breakage of inlet/outlet piping from uplift
- Tearing of the tank wall at discontinuities
- Tearing of tank wall from overconstrained stairways between the foundation and tank shell
- Roof damage caused by sloshing
- Foundation failures and liquefaction

Schiff (1991) presents a summary of observed damage to tanks. Flatbottom vertical tanks tend to be most vulnerable to earthquakes, especially tanks with a large liquid depth to radius ratio. One failure mode of the tank is buckling and is caused by rocking of the tank or differential settlement of the foundation under the tank Unanchored tanks with a radius-to-wall thickness of over 600 have been damage most often. These tanks develop sufficient overturning moment to cause the edge to lift off the ground. The opposite side sustains high compressive stresses which cause bulging at the base. Summers (1997) reports that a 100-foot diameter tank 30 feet high sustained 14 inches of uplift in the 1971 San Fernando earthquake. Similar tank behavior was noted in the 1989 Loma Prieta earthquake with an observation of 6 to 8 inches of uplift and 18 inches in the 1964 Alaska earthquake. Even anchored tanks can fail in this manner if there is anchorage failure. Generally a pattern of well distributed anchor bolts works best compared with fewer larger bolts. Maintenance is requirement to inspect the condition of the anchor bolts and replace those with corrosion.

Vertical motion can cause local tensile membrane deformation, elephant foot bulging, at the base of the tank. This can also be induced by rocking. It is interesting to note the annular volume of the bulge is about equal to the earthquake vertical displacement times the tank area. It is postulated that the fluid has high inertia and the increase in fluid pressure from the vertical component of the earthquake causes the perfectly symmetrical bulge. Increasing the wall thickness may reduce the occurrence, but might simply result in the buckling occurring high up on the tank. The weld between the tank base plate and side wall has also been observed to fail. This is caused by uplift forces and is often associated with corrosion induced weakness. A failure of the weld can open a portion of the seam causing rapid loss of contents and a partial vacuum in the tank causing internal buckling. Tank venting is important to restrict implosion.

Small unanchored tanks less than 30 feet in diameter have been observed to slide on their foundation. In tanks which are full, sloshing can cause roof and upper wall failures. As noted liquefaction was a major cause of the extent of waterfront damage and can cause settlement and lateral spreading.

The primary failure mode of horizontal tanks is anchorage failure or inadequate anchorage which causes tank slippage off the saddles. Typically the tank is fully anchored only on one side to allow for expansion. The single restraint must be capable of withstanding horizontal, vertical and torsional components of motion. The saddle must be designed to resist forces acting on its weak axis as well. Elevated fuel tanks often fail by buckling of the supports. These tanks stands require adequate tie-down and diagonal bracing

Water tanks tend to be kept full however hydrocarbon tanks tend to be half-full and sloshing must be considered. Lack of tank venting has resulted in implosion. Anchor bolts embedded in concrete used for tank uplift restraining must have sufficient concrete confinement to prevent pullout. Shear reinforcing should be used to provided needed concrete confinement to prevent anchor bolt failure. Typically anchor bolts for new construction are designed with a safety factor of 4; a value of 3.0 is used for evaluation of existing anchors. Provisions must be made to evaluate the effect of corrosion in reducing the strength of existing construction.

To achieve the required system performance and satisfy regulations additional backup hazardous material containment systems are used. Containment systems are composed of either a singular system or a dual system as mandated by public law discussed in the Criteria. A singular system provides only a single structural element system for material containment. Singular systems are restricted to small systems of less than 660 gallons such that a failure shall not produce catastrophic damage. A dual system is composed of a primary containment structure and a secondary containment system which shall function should the primary system be damaged. Containment systems open to rain shall need to be drained.

Design of tanks shall utilize the API 650 procedures discussed above.

Tanks shall be designed against sliding and uplift and be fully anchored. The height of sloshing may be calculated using an equation by Wozniak and Mitchel (1978). This height should be used for freeboard calculations associated with roof damage. The hydrodynamic forces which create overturning moments also act on the foundation and must be taken into account in foundation design.

Essential tanks shall be designed to resist Level 3 earthquakes using response spectra and the API 650 procedures.

For both ordinary and essential tanks, a requirement exists to prevent uncontrolled loss of contents and pollution of the environment an event for a Level 4 event. Such requirements shall be met by provision of a containment system. Singular systems must be designed so that the structure itself provides the margin of safety to preclude release of materials. Dual systems may be evaluated on the basis of total system performance allowing for the presence of the secondary confinement, such that any release from the primary containment is confined within the secondary containment. The secondary containment must function at such a level so as not to permit an unacceptable release of materials.

Failure of pipe to tank connections is common when there is insufficient flexibility to accommodate differential motion between the tank and pipe network. This can be prevented by having the first pipe anchor point at a sufficient distance (15 pipe diameters minimum) from the edge of the tank and the pipe oriented in a radial direction away from the tank. Flexible connections and expansion joints can accommodate differential motion provided they are sized properly. The most important element of seismic design of pipelines is proper siting. It is imperative to avoid areas of lateral spreading and landslide. Additionally stairways should not be attached to both the foundation and the tank wall.

Summers (1997) presents the following information:

> Tank uplift during earthquakes can damage attached piping and other appurtenances. ...... anchored tank appurtenances may he designed for some level of anchor bolt stretch. A value of 2 inches is proposed in the latest NEHRP provisions (BSSC, 1994).
>
> API 650 states that piping attached to the tank bottom that is not free to move vertically shall be placed a radial distance from the shell/bottom connection of 12 inches greater than the uplift length predicted by the API 650 uplift model. The API 650 uplift model, however, may underpredict the amount of radial uplift (Manos, 1987; Dowling and Summers, 1993). It may be prudent to consider changing this requirement to... twice the API 650 model.......
>
> Walkways between tanks should he designed to accommodate relative movement of the tanks. ..... In lieu of a more rigorous analysis, a walkway should he designed to accommodate a total of 12 to 18 inches of movement, at least in the zones of high seismicity.

Attached ringwalls should be designed appropriately. Anchoring a tank to a small ringwall and not developing the forces into the soil by the weight of the ringwall or with piles should he viewed with caution. Anchor bolts need to be designed such that they behave in a ductile manner, both in terms of the force transfer to the shell and pullout from the concrete foundation.

Existing Tanks

Tanks built prior to the late 1970s probably lack consideration of seismic design since it was during that period in which code provisions were first implemented. However the provisions in API and AWWA are generally thought to be conservative such that evaluation of existing tanks by the new tank criteria may unduly penalize them. Summers (1997) reports the following:

> There are several alternatives to the API methodology that might be considered for use in evaluation of existing tanks. One such method is a modified version (Dowling and Summers, 1993; Summers and Hults, 1994; and ASCE Task Committee on Seismic Evaluation and Design of Petrochemical Facilities, 1997) of a method developed by George Manos (Manos, 1987) presented herein Manos' method is based on experimental studies, as well as on observed behavior of unanchored tanks during past earthquakes. Instead of trying to model the complex uplifting plate behavior, Manos assumes a stress distribution at which the shell will buckle and solves for the resisting moment produced by the sum of the stresses. This resisting moment can then be compared to the overturning moment and the resisting acceleration solved for.
>
> The method proposed herein for evaluation of unanchored storage tanks is based on that of Manos, but includes some important variations. The most notable of these are (Dowling and Summers, 1993):
>
> a. Tank anchorage is recommended in zones of high seismicity whenever the ratio of safe operating height to tank diameter exceeds two. Based on the data presented in Manos, and the higher level of risk for taller tanks, this is believed to be the upper limit of applicability of the Manos method.
>
> b. The allowable compressive stress in the tank shell should not exceed 75% of the theoretical buckling stress, as presented in Manos, nor should it exceed the material yield strength. This last requirement is significant for thicker-walled tanks. Note that an increase in the allowable compressive stress beyond 75% of the theoretical buckling stress may be justified under certain circumstances.

c. The compressive force in the tank shell should not exceed the total weight of the fluid contents. This has the effect of imposing an upper bound on the resisting moment.

A comparison of the results of an evaluation of a 35 ft diameter, 30 ft high tank in a high seismic zone filled to a height of 26 ft 4 in, using the modified Manos and API methodologies, ... (was made. The) API approach would require either a reduction in fill height by about 40% to 16 ft 6 in or tank anchorage, whereas the modified Manos method indicates that the seismic safe operating height can be increased to 20 ft 1 in. Hence, the required reduction is reduced from 9 ft 10 in to 6 ft 3 in, and the benefit is immediately apparent.

The Manos (1987) develops the following relationships for the compressive member stress distribution near the tank bottom as:

$$\sigma_{max} = 0.75 \quad _{cl}$$

where

$$\sigma_{cl} = \frac{E\ t_s}{R\sqrt{3(1-\upsilon^2)}}$$

$$\sigma = \sigma_{max} \cos\left(\frac{\pi\phi}{2\phi_0}\right)$$

if = 0.3

$$\sigma = 0.46\frac{Et_s}{R} \cos\left(\frac{\pi\phi}{2\phi_0}\right)$$

$$\phi_0 = 0.65\ S\left(\frac{R}{H}\right)^n\left(\frac{t_s}{t_p}\right)^{0.1}$$

$$n = 0.1 + 0.2\frac{H}{R} \leq 0.25$$

where

E Young's modulus
H Liquid height
R Tank radius
S Foundation coefficient

t_s tank wall thickness
t_p tank bottom plate thickness
υ Poisson's ratio
σ axial member stress
σ_{cl} Theoretical buckling stress

Manos develops an empirical expression for the limiting impulsive acceleration capacity of a tank, C_{eq}, as

$$C_{eq} = \frac{0.372}{\rho_w} \frac{S\,E\,t_s^2}{G\,R\,H^2} \frac{m_t}{m_1} \left(\frac{R}{H}\right)^n \left(\frac{t_s}{t_p}\right)^{0.1}$$

where

G Liquid density ratio
m_1 Liquid impulsive mass
m_t Total liquid mass
ρ_w Density of water or tank liquid

Manos compares the acceleration capacity to the applied acceleration which is based on a tank response spectrum determined from an amplified ground motion spectrum between the periods of 2 and 9 seconds having 2 percent damping. He proposes a 4.3 acceleration amplification factor to be applied to the ground motion spectrum as a conservative approximation of structure amplification. The acceleration capacity must be larger than the acceleration demand.

Special Drainage for Petroleum Offloading and Fueling Piers.

The following requirement shall be adhered to pertaining to special drainage for petroleum offloading and fueling piers:

a) An intercept system shall be required to collect oil spills. In normal operation, deck drainage shall outfall through the sump pumps into the harbor. If an oil spill occurs, pressing a deck-mounted button shall close a motor-operated outfall valve and start the sump pumps which pump the spill to a collection point. when the spill drainage procedure is completed and all oil is removed from the system, the system shall revert to normal operation.

b) Contaminated rainwater runoff of all deck drainage due to contact with residual drippings on the deck shall be collected.

Utilities On Piers

Piers may contain pipelines for freshwater, saltwater, steam, compressed air, waste oil, sewer, fuels, as well as electrical power and communication lines. Ship demands dictate the configuration. In general design of these lines follows the general provisions discussed herein. It is essential that the lines be attached to the supporting structure with sufficient rigidity that the lines are restrained against independent movement. Attachments to a pier may be analyzed as simple two-degree-of freedom systems as discussed in NAVFAC P355, Chapter 12. Resonance amplification can occur when the natural period of the supported pipe is close to the fundamental period of the pier structure. Flexible connections/sections shall be used to bridge across expansion joints or other locations where needed. All piping and utility lines on a pier shall be designed as essential construction. Specifically, the provisions of NAVFAC P355 Section 12-7d shall be used. Section 12-7d is discussed above under pipelines. Pipelines containing hazardous materials may have to be of double wall construction based on requirements of local environmental requirements. Check-valves should be used to minimize the loss of contents to minimize the size of a spill if there is a pipeline break.

Electric Power

A typical electric power system includes transformer and distribution lines, local transformers and backup generators. Linkages exist between electric power and other lifelines; for example, electricity is needed for pumps to maintain pressure in water distribution systems. Most damage has resulted from overturning or sliding of inadequately anchored or braced components. Often electrical equipment is situated on top of poles or supports undergoes extensive displacements rupturing attached cables. Pole mounted transformers are supported on raised platforms; typically they are not secured to the platform.

Inadequately mounted transformers have been observed to fall from pedestals causing major damage to bushings, radiators and internal parts. An alternative failure mode is excessive sliding without overturning. Sliding breaks the rigid bus connections, the lightning arrestors and insulators. Past practice had transformers mounted on rails anchored in concrete slabs. When these mounts failed, extensive damage resulted. Schiff (1991) suggests that new installation design use concrete pads with steel anchor plates securely embedded in the pad and flush with the surface. The transformer is welded to the anchor plate eliminating the need for an intricate pattern of tie down bolts. Spare transformers were kept unsecured for relocation as needed, which can overturn. All transformers whether in service or spare require the same restraint.

Criteria for electrical power lifelines focuses on providing adequate anchorage. All transformers on poles or platforms shall be anchored against overturning or sliding. All equipment shall be anchored as required. Equipment deemed as of ordinary importance shall have lateral force requirements based on provisions of the 1994 Uniform Building Code and NAVFAC P355. Equipment deemed as essential shall have the lateral force requirement based on local site conditions using peak ground acceleration for essential level facilities and a response spectra. In any case lateral forces shall not be less than Code provisions with an importance factor for essential structures/components . This resulting force shall be used as a substitute for Code forces and all remaining Code provisions will apply.

Telecommunications Lifelines

Telecommunications encompasses conventional telephone requirements, communication systems, and equipment control lines. The equipment must be rugged enough to withstand the shaking. The IEEE has established fragility requirements for some equipment found in nuclear power plants. Some other types of equipment also have fragility data. The equipment must be attached in a manner to prevent damage. Attachment can be made by rigidly securing the item against overturning and sliding or where the equipment is delicate it may be mounted on isolators to reduce transmitted motions. A variation of both approaches consists of leaving a large piece of equipment free to slide within restrained limits. This limits the shaking motion which can be transmitted to the equipment by allowing sliding to occur; elastic bumpers limit the range of motion. Obviously the equipment must have an aspect ratio to preclude overturning.

Traditional damage has included overturning of cabinet mounted electronics, failures of suspended ceilings, rupture of piping and water damage to equipment, rupture of cables connecting equipment which became dislodged, weld failures, and inadequate sizing of restraints. Design calculations must consider the inertia force of an object in overturning and sliding. Elements attached to the structure must consider the relative displacement between anchorage points. Flexible supports must consider resonance points where the period of vibration of the flexible mount is the same as that of the structure; stiffening the mount can eliminate resonance.

Required support equipment generally includes backup power generators, uninterruptible power supplies, emergency lighting, voltage controllers, etc. and may also include air-conditioning units, halon firefighting systems etc.

Calculation Of Lateral Force Requirements

The 1997 NEHRP FEMA 273 provisions calculate seismic forces as:

$$F_p = 1.6\, S_{XS}\, I_P\, W_p$$

Alternatively F_p may be calculated by:

$$F_p = 0.4\, a_p\, S_{XS}\, I_P\, W_p\, (1 + 2x/h) \;/\; R_p$$

F_p calculated by the second equation need not exceed F_p calculated by the first equation but a minimum F_p is given by:

$$F_{p\ minimum} = 0.3\, S_{XS}\, I_P\, W_p$$

where:

a_p Component amplification factor that varies from 1.00 to 2.50 (select appropriate value from FEMA 273 Table 11-2).

F_p Seismic design force centered at the component's center of gravity and distributed relative to component's mass distribution.

S_{XS} Spectral response acceleration at short periods

I_p Component importance factor that is either 1.00 or 1.50

W_p Component operating weight.

R_p Component response modification factor that varies from 1.25 to 6.00 (select appropriate value from FEMA 273 Table 11-2).

x Elevation in structure of component relative to grade elevation.

h Average of elevation of structure relative to grade elevation.

The force F_p shall be applied independently vertically, longitudinally, and laterally in combination with service loads associated with the component. When positive and negative wind loads exceed F_p for nonbearing exterior wall, these wind loads shall govern the design. Similarly when the building code horizontal loads exceed for interior partitions, these building code loads

Evaluation Of Above Ground Piping Systems

The following is taken essentially verbatim from “Proposed Guidance for California Accidental Release Prevention Program Seismic Assessments” (1998).

Evaluation of piping systems are primarily accomplished by field walkthroughs. Such qualitative evaluations of piping systems are best done by an engineer experienced in this area, visually inspecting the piping system under concern. This is preferred because some piping is field routed and in some instances, piping and supports have been modified from that shown on design drawings. This guidance is primarily intended for ductile steel pipe constructed to a national standard. Evaluation of other piping material is also discussed below.

The procedure for evaluating above ground piping systems should be as follows:

1) Identify piping systems to be evaluated.

2) Determine original design code basis and materials of construction, to the extent possible.

3) Assess extent of obvious corrosion/erosion.

4) Perform a walkthrough of the piping systems for seismic capability. Document the walkthrough and identify areas for detailed evaluation.

5) Complete the detailed evaluation of any identified areas and recommend remedial actions.

Damage to or failure of pipe supports should not be construed as a piping failure unless it directly contributes to a pressure boundary failure. The intention here is to preserve the essential pressure containing integrity of the piping system but not necessarily leak tightness. Therefore, this procedure does not preclude the possibility of small leaks at bolted flanged joints. Ductile piping systems have, in general, performed adequately in past earthquakes. Where damage has occurred, it has been related to the following aspects of piping systems:

1) Excessive seismic anchor movement.

2) Interaction with other elements.

3) Extensive corrosion effects.

4) Non-ductile materials such as cast iron1 fiberglass (PVC), glass, etc. combined with high stress or impact conditions.

Seismic anchor movements could result in relative displacements between points of support/attachment of the piping Systems. Such movements include relative displacements between vessels, pipe supports, or main headers for branch lines. Interaction is defined as the seismically induced impact of piping systems with adjacent structures, systems, or components, including the effects of the falling hazards. Corrosion could result in a weakened pipe cross section that could fail during an earthquake. Additional aspects of piping systems which should also be reviewed during the walkthrough for seismic capability are:

1) Large unsupported segment of pipe,

2) Brittle elements,

3) Threaded connections, flange joints, and special fittings, and

4) Inadequate supports, where an entire system or portion of piping may lose its primary support.

Special features or conditions to illustrate the above concerns include:

1) Inadequate anchorage of attached equipment,

2) Short/rigid spans that cannot accommodate the relative displacement of the supports, e.g., piping spanning between two structural systems,

3) Damaged supports including corrosion,

4) Long vertical runs subject to inter level drift,

5) Large unsupported masses (e.g., valves) attached to the pipe,

6) Flanged and threaded connections in high stress locations,

7) Existing leakage locations (flanges, threads, valves, welds),

8) External corrosion,

9) Inadequate vertical supports and/or insufficient lateral restraints,

10) Welded attachments to thin wall pipe,

11) Excessive seismic displacements of expansion joints,

12) Brittle elements, such as cast iron pipes,

13) Sensitive equipment impact (e.g., control valves), and

14) Potential for fatigue of short to medium length rod hangers which are restrained against rotation at the support end.

The walkthrough is the essential element for seismic evaluations of piping systems. Careful consideration needs to be given to how the piping system will behave during a seismic event, how nearby items will behave during a seismic event (if they can interact with the piping system) and how the seismic capacity will change over time. The walkthrough should be performed by a licensed engineer familiar with how equipment responds to earthquake loads. Detailed analysis of piping systems should not be the focus of this evaluation. Rather it should be on finding and strengthening weak elements. However, after the walkthrough is performed and if an analysis is deemed necessary, the following general rules should be followed:

1) Friction resistance should not be considered for seismic restraint, except for the following condition: for long straight piping runs with numerous supports, friction in the axial direction may be considered,

2) Spring supports (constant or variable) should not be considered as seismic supports,

3) Unbraced pipelines with short rod hangers can be considered as effective lateral supports if justified,

4) Appropriate stress intensification factors ("i" factors) should be used, and

5) Allowable piping stresses should be reduced to account for fatigue effects due to significant cyclic operational loading conditions. In this case the allowables presented in the next section may need to be reduced.

6) Flange connections should be checked to ensure that high moments do not result in significant leakage.

Procedures for interaction evaluation of piping are as follows:

1) Regulated Substance (RS) piping should be visually inspected to identify potential interactions with adjacent structures, systems, or components. Those interactions which could cause unacceptable damage to piping, piping components (e.g., control valves), or adjacent critical items should be mitigated.

Note that restricting piping seismic movement to preclude interaction may lead to excessive restraint of thermal expansion or inhibit other necessary operational flexibility.

2) The walkthrough should also identify the potential for interaction between adjacent structures, systems or components, and the RS piping being investigated. Those interactions which could cause unacceptable damage to RS piping should be mitigated Note that falling hazards should be considered in this evaluation.

Procedures for corrosion evaluation of piping are as follows:

1) During walkthrough identify conditions conducive to external corrosion.

2) Wall thickness should be evaluated for potential reduction due to erosion or corrosion.

3) Extent of internal corrosion/erosion can be evaluated by any of the following methods:

a. Review of existing corrosion inspection program for RS piping systems,
b. Review of successful operating experience, or
c. Wall thickness measurements.

4) Compare existing corrosion experience and anticipated corrosion to original design corrosion allowance.

The reader is should consult the “Proposed Guidance for California Accidental Release Prevention Program Seismic Assessments” (1998) for additional material not included here such as support design and inertia loads.

Terminal Inspection Team Assessment

A terminal inspection should include a focused study on the relationship of the oil supply system and facility operational and safety requirements. A first step identifies criteria for determining critical functions that require secure energy and selecting those facilities supporting the required safety/operational functions

Post-earthquake recovery efforts must focus on life safety, disaster control and sustaining required operation of essential functions. Recovery efforts must be prioritized to maintain required operations and minimize further damage. It is also essential to relate the repair of utility systems to facility needs since experience has shown that utility system disruptions can produce major impacts upon essential functions.

The elements of the terminal lifeline assessment procedure are to:

Form an Assessment/Mitigation Team
Gather information about lifeline utilities
Determine essential-function requirements from users
Consider utility outage scenarios
Assess the vulnerability of required utility lifelines
Develop mitigation measures

The most significant step in conducting a lifeline assessment is understanding what are the critical operations of the terminal. What critical facilities support these operations and what utilities are critical to support the critical operations? In addition to safe operational requirements are the disaster control and recovery functions which are also highly dependent on lifelines.

The utility/lifeline assessment team must contain a technologist with the ability to understand and assess the terminal’s utilities. The team will also need accurate utility system drawings and diagrams for each system being evaluated. Operators and maintenance personnel can often identify vulnerable points for critical systems. Reports from utility studies, communication system studies, utility contingency plans, security and other plans and studies may provide drawings and analysis that are applicable to this effort. Key hardware components servicing each essential operation need to be identified. Key hardware components are pieces of equipment which must remain operational to support the operation and represent critical links. Utility outage scenarios must be developed based on the seismic potential and the key hardware components accessibility and fragility. The effects of the utility outages must be analyzed including the evaluation of the response, repair, and recovery capability of the oil terminal. Once the team has accomplished the utility systems assessment, they can identify the corrective measures necessary to remedy deficiencies. These remedies are likely to include projects and/or activity operational procedures. The team should rank the mitigation measures and develop plans to implement them.

Analysis

The approach outlined in this section is a guide to assessing the vulnerability of those control systems and utilities which provide service to essential oil terminal operations. The survey team should examine fire detection and suppression, electricity, water, thermal, sanitary and industrial wastewater, compressed air, and communications as well as any other utility. The information gathered is meant to stimulate the thought process and is a tool to assess the utility systems. In gathering these data one will not only compile a comprehensive source of valuable utility information, but will also discover information gaps which might prove critical in an actual utility outage situation. The vulnerability assessment report produced from this information will require rigorous analysis of the particular lifeline utility systems and operating procedures. In analyzing a utility system, it is best to follow a logical pattern from the point of utility supply through the onbase distribution system to the final end-user. In the assessments, provide simplified schematics of each utility system which have key components.

For each outage scenario considered, both area-wide and local utility outages, the impact upon the essential operations should be assessed, the minimum utility requirements needed to support these operations should be determined, and possible corrective measures to meet these requirements should be evaluated. Consideration should be made regarding the interrelationships between utility systems.

Response, Repair, and Recovery Capability

For every key component identified, a comprehensive evaluation the ability to restore the item to affected areas should be conducted. For example, restoration of power may include repair to the disabled component, replacement of the disabled component, actual bypass of that component or provision of backup power. The potential for subsequent problems from the repair, replacement, or bypass should be part of the overall evaluation. Items that should be considered include the following:

1. Availability of spares and replacement equipment (location, time, administrative procedures)
    a. Onsite
    b. Commercial utility
    c. Commercial suppliers
    d. Other sources

2. Availability of equipment needed to effect repairs

a. Heavy transport equipment
b. Trucks
c. Special tools and machinery

3. Availability of personnel

a. Activity Maintenance Personnel

1. Crew skill mix
2. Level of training
3. Degree of experience
4. Knowledge of installation systems
5. Multiple assignments and responsibilities
6. Staff reassignment

b. Commercial contractors

1. Number of contractors available and past relationships
2. Formal agreements or contracts
3. Administrative and financial limitations

c. Commercial utility

1. Availability potential
2. Formal and informal agreements
3. Other commitments or obligations

4. Consideration of adverse working conditions and circumstances

5. Implementation of load shedding/conservation to match available supply

6. Availability of backup generator sets

a. Verification of operation of generators
b. Maintainability/ability to operate for extended periods
c. Refueling requirements and procedures
d. Ability to relocate and connect to loads

Key Hardware Components

Electrical Distribution System

a. Commercial feeds to activity and their point of origin, point of connection to terminal, and capacities
b. Backup generators

    1. Number of fixed and portable sets assigned
    2. Other generators available on the installation
    3. Size, age, and present assignments

c. All essential loads and all key components utilizing a current wiring diagram
d. Load shedding and any other relevant contingency plans

Water Distribution Systems

Water system key hardware components must include potable water, fire protection water and water requirements related to thermal energy systems (makeup water for boilers and cooling towers).

a. All essential loads and key components shown on current drawings
b. Commercial lines serving the activity and the points of origin
c. Onsite water sources (if any)
d. Onsite water treatment facilities (if any)
e. Capacity and location of storage facilities (if any)
f. Key components dependent on electrical power

    1. Pumps
    2. Water treatment equipment
    3. Valves
    4. Controls

g. Backup electrical generator sets

    1. Number of fixed and portable sets assigned
    2. Size, age, and present assignments

h. Availability of water transport
i. Treatment chemical requirements

Wastewater (Sewage/Industrial Systems)

Wastewater system key hardware components should include collection and treatment facilities for domestic sewage and industrial wastewater. Consider the possible creation of hazards associated with mixing incompatible industrial wastewater streams. Additionally, repairs to sanitary sewers, where waste may be septic, should be accomplished with protective gear and respiratory equipment.

a. Public lines serving the terminal and their point of origin
b. Generating activities
c. Onsite treatment facilities (if any)
d. Storage capacity (i.e., 55 gallon drums, tankers, holding tanks, etc.)
e. Critical functions which generate wastewater
f. Key hardware components from point of generation to treatment/disposal
g. Key components dependent on electrical power

    1. Pumps
    2. Wastewater treatment equipment
    3. Valves
    4. Controls

h. Backup electrical generator sets

    1. Number of fixed generator sets
    2. Size, age and present assignment

i. Availability of transport (i.e., tankers, barges etc.)
j. Treatment chemical requirements

<u>Compressed Air</u>

a. Essential functions requiring compressed air (if any)
b. Sources of compressed air (central, point of use)
c. Key components dependent on electric power
d. Identify compressors capable of operating independent of electrical power
e. Isolation valves
f. Cross connects between distribution lines
g. Backup compressors at critical points of use
h. Backup electrical generator sets

CHECKLIST FOR WALK-THROUGH SCREENING

The Army Corps of Engineers sponsored a study of lifelines on military bases. One of the products which came out of that work was a set of checklists for screening water supply lifelines part of which is reproduced below.

Pump Stations

T F **PIPING PENETRATIONS:** Piping at wall penetrations and equipment has flexible connections or sufficient clearance.

T F **ANCHORAGE:** Pumps, motors, control cabinets, generators and controls are adequately anchored.

T F **VIBRATION ISOLATORS:** Vibration isolated pump and drive units have seismic snubbers to limit motions.

T F **OFF-SITE POWER:** Off-site electrical power or has backup provisions.

Process Tanks And Structures

T F ANCHORAGE: Tanks are adequately anchored.

T F IMMERSED COMPONENTS: Concrete or timber baffles, rotating equipment, and other immersed components have been designed for sloshing and inertial effects.

T F PIPING PENETRATIONS: Tanks have flexible connections at piping penetrations.

T F LIQUEFACTION: Structures are not buried in liquefiable soil.

Equipment and Piping

T F ANCHORAGE: Plant equipment is adequately anchored.

T F COMMON FOUNDATIONS: Pumps and motors are on common foundations.

T F PIPING CONNECTIONS: Flexible piping connections are used on all equipment.

T F EXPANSION JOINTS: Piping which crosses expansion joints has flexible connections.

T **F** **BRACING:** All piping runs are transversely and laterally braced.

T **F** **HAZARDOUS MATERIALS PIPING:** All piping runs are transversely and laterally braced. Provisions for containment are present in the event of a breach.

T **F** **FALLING DEBRIS:** Falling debris cannot damage yard and plant piping.

T **F** **HORIZONTAL TANKS:** Horizontal tanks (including fuel, liquid natural gas, propane, diesel, chemical) are adequately anchored.

T **F** **ELEVATED TANKS**: Elevated tank and equipment legs are adequately braced.

Pipelines

T **F** **BACKFILL AND BEDDING:** Pipes are buried in compacted bedding and fill.

T **F** **COUPLINGS**: Couplings are flexible with rubber gaskets.

T **F** **MATERIALS**: Pipes are constructed from appropriate materials

T **F** **FAULT CROSSING**: Pipes do not cross active earthquake fault zones.

T **F** **ELEVATED PIPES:** Elevated pipes are braced for longitudinal and transverse movements.

T **F** **PIPING PENETRATIONS:** Pipes have clearance and flexible couplings at wall penetrations.

T **F** **CORROSION:** Internal and external corrosion has been studied and does not affect seismic performance.

Storage Tanks

T **F** **SEISMIC SHUT-OFF:** There is an automatic earthquake-triggered shut off valve.

T **F** **PIPING PENETRATIONS:** Piping connections have seismic joints which allow rotation and axial movement.

T **F** **ANCHORAGE:** Steel tanks are anchored.

T F **ANCHOR EMBEDMENT:** Anchor bolt and strap embedment will develop yield strength.

T F **ANCHOR DUCTILITY:** Anchor bolts and straps have at least 6 inches stretch length above the foundation.

T F **WIRE WRAPPED TANKS:** Wire-wrapped concrete tank reinforcing is not corroded.

T F **SLOSHING:** Roofs and supporting columns are designed to resist the effects of sloshing water.

T F **FOUNDATIONS:** Differential settlement, liquefaction, landslides or fault rupture are not expected at this site.

T F **WELD CORROSION ALLOWANCE:** Steel tank weld thickness was increased to allow for corrosion.

T F **TANK BRACING:** Elevated tank legs are braced.

T F **COMPRESSION BRACING:** Elevated tank leg bracing has significant compression capacity.

Containment Reservoirs for Tanks

T F **LIQUEFACTION:** Earth berms will not liquefy.

T F **LINING**: The reservoir is lined.

T F **SEISMIC SHUT-OFF**: There is an automatic earthquake-triggered shut off valve.

Lifeline Support Buildings

T F **BUILDINGS:** Buildings have been evaluated and found acceptable according to FEMA procedures.

T F **EXITS**: Suspended equipment over exit corridors is has adequate lateral bracing and vertical support.

T F **EXHAUST FANS**: Failure of exhaust fans will not create areas with a hazardous atmosphere.

T F **ANCHORAGE**: Office and lab equipment is adequately anchored.

T F **COMPUTER FLOORS:** Computer floor pedestals are braced along every grid line. Pedestals and braces are bolted to the floor.

Electrical Power

T **F** **OFF-SITE POWER:** Failure of off-site electrical power will not affect operations.

T **F** **ANCHORAGE:** Transformers, control cabinets, switchgear, motor control centers, etc. are adequately anchored.

T **F** **POLE MOUNTED TRANSFORMERS:** Pole mounted transformers are laterally braced and anchored.

Uninterrupted Power Supply

T **F** **ANCHORAGE:** Charger and invertor units are anchored.

Emergency Power Engine Generators

T **F** **ANCHORAGE:** Generator is bolted to the floor.

T **F** **VIBRATION ISOLATORS:** Isolators and retainers are not cast iron.

T **F** **SNUBBERS:** Vibration isolators have seismic snubbers.

T **F** **SUPPLY LINES:** Fuel, electric**,** cooling water, air start, exhaust and water lines can accommodate relative movement.

T **F** **FUEL TANKS:** Fuel tanks are adequately braced and anchored.

T **F** **COMMON FOUNDATIONS:** Engines and generators are on common foundations.

T **F** **DIESEL FUEL:** Diesel fuel is changed at least once per year to prevent clogged fuel filters and injectors.

T **F** **COOLING SYSTEM**: Cooling system does not leak and has enough make-up water.

T **F** **SYSTEM LOADS**: System loads have not increased since the generator was installed.

T **F** **AIR START:** Air start system compressor and air tanks are adequately anchored.

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